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REINFORCED CONCRETE PILING

REINFORCED CONCRETE PILING

BY

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AND

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BOOKS ON CONCRETE

A list of other practical and useful books on concrete and reinforced concrete design and construction, precast concrete, cement, and allied subjects is given on page xviii.

PREFACE

IN this book the authors have aimed at describing current British practice in designing, manufacturing, and driving reinforced concrete piles. The work is presented in a concise form and as far as possible is based on practical experience. Since piles are frequently used as members of braced frames and in the construction of retaining walls, this part of the subject is dealt with at length.

Modern methods of manufacturing and driving piles are illustrated in detail, and a chapter is devoted to cast-in-situ piles.

To assist engineers designing piled foundations, tables are included which give the safe short-column load and moment of resistance of many typical square and octagonal sections. The greatest lengths of these piles that can be lifted at their ends without exceeding the safe moments of resistance are also tabulated. We are indebted to Mr. C. L. T. Griffith for his assistance in making the calculations on which these tables are based, and to other engineers for their collaboration in the preparation of the book.

F. E. W.-S.
W. S. G.

February 1938

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CHAPTER I

INTRODUCTORY

Advantages of Concrete Piles.

It may fairly be said that concrete piles are steadily superseding timber piles in most countries, and this for several good reasons. Concrete piles are on the whole more durable than timber, especially in ground which is damp or in sea water. Provided a timber pile is wholly driven into earth which is below ground water level it may have a very long life, a well-known example being the timber piles in the foundation of the campanile of St. Mark's, Venice, which were found to be in such a good state of preservation after the lapse of a thousand years that the engineers responsible for the reconstruction of the tower left them in position. On the other hand there are very many examples in which the piles were only partly below the ground water line and suffered severe decay, whereas well-made concrete piles can be relied on not to decay in any position.

All types of reinforced concrete piles are easily bonded to the superstructure by stripping the concrete heads and bending the reinforcement into pile caps or continuous beams connecting the heads of adjacent piles, thus increasing the resistance of the foundation to lateral movement. The pile caps in this case, being of reinforced concrete, are of small volume and bring very little extra load on to the piles.

Reinforced concrete piles are easily lengthened by stripping the heads, lapping additional bars to the exposed reinforcement to secure continuity, erecting shuttering as for a concrete column, and placing the additional concrete.

Unlike timber piles, reinforced concrete piles can be quickly obtained of any required length and cross section. Among pre-cast reinforced concrete piles of noteworthy dimensions which have been driven are the following :

Locality and Date of Driving.	Size.
Auckland, New Zealand (1906)	100 ft. by 20 in. by 20 in.
San Francisco Harbour (1913)	91 ft. by 20 in. by 20 in.
Newport News, U.S.A. (1928)	115 ft. by 24 in. by 24 in.
Manilla (1925).	110 ft. by 24 in. by 24 in.
Lake Pontchartrain, U.S.A. (1927).	75 ft. to 80 ft. by 24 in. by 24 in.
Halifax, Nova Scotia (1915)*	75 ft. to 77 ft. by 24 in. by 24 in.
Renfrew, Glasgow (1931)	93 ft. by 17 in. octagonal.
North Shields (1937)	89 ft. by 24 in. octagonal (hollow).

* Each pile of the group at Halifax carries 100 tons.

Where piles can be used in bridge construction they are almost always cheaper than compressed-air foundations. Practically the only exception is where the

river bed is very easily eroded and where therefore a deep heavily-timbered excavation must be removed before reaching the level at which the pile heads are required to finish. In this case the deep monolith or caisson type of foundation is necessary in order to avoid undermining.

Materials.

CEMENT.—If Portland cement is used, it should be obtained from an approved British manufacturer and conform to all the requirements of the current British Standard Specification for Portland Cement. Quick-setting cement should not be used. Each consignment should be accompanied by a certificate from the manufacturers giving the results of the tests that have been made on it and stating the place and date of manufacture. On important work independent tests of the cement should be made by an experienced tester.

So far no standard specification for rapid-hardening Portland cement has been published, but this cement should also comply with the provisions of the latest edition of the British Standard Specification for Portland Cement. In addition, the residue on the 180-sieve (32,400 meshes per square inch) should not exceed 1.5 per cent., and cement and sand briquettes made as described in the Specification should sustain a tensile stress of at least 300 lb. per square inch when twenty-four hours old and 550 lb. per square inch when seven days old.

A standard specification for aluminous cement has not yet been prepared, and it is therefore the more important to obtain it from a first-class manufacturer. When aluminous cement is used the extra cost of the cement is an important factor, and may not be justified unless the piles are urgently required or unless an equal or greater amount can be saved on shuttering, hire of pile frame, etc.

The principal advantages arising from the use of rapid-hardening Portland and aluminous cements are a saving in shuttering and in materials and space occupied by the casting platform; also it is possible to release them from the shuttering and drive them at an earlier date. Before the introduction of rapid-hardening cements it was often the custom to allow a period of two months to elapse between the times when piles were manufactured and driven. As a result foundation work was often delayed because after driving the first piles it was discovered that the length was insufficient or too great. By using rapid-hardening cements the piles mature quickly and the percentage loss due to wrong lengths can be reduced without delay by making smaller quantities at first and altering the lengths if driving proves that this is necessary.

Aluminous cements and Portland blast-furnace cements are reputed to be highly resistant to sea action, but this is only true if the concrete made from them is rich and dense.

STEEL.—The reinforcement used in piles in Great Britain is mild steel round bars of quality A as defined in and complying with the requirements of the British Standard Specification (No. 15) for Structural Steel. For very heavily-loaded piles mild steel complying with this specification and, in addition, having a yield point of 44,000 lb. per square inch may be used, but it is doubtful whether there is any advantage in using this material.

AGGREGATES.—The coarse aggregate may be either round pebbles or crushed stone, provided that it is hard, strong, and perfectly clean and free from clay.

Its size should be between $\frac{3}{4}$ in. and $\frac{3}{16}$ in. and it should be evenly graded between those limits.

The fine aggregate or sand may also be either round or angular, provided that it is hard and strong and perfectly clean. Its size should be from $\frac{3}{16}$ in. downwards. It should be well graded and should not contain very fine material, for example, such as would pass a 60 B.S. sieve ($\frac{1}{100}$ in.).

It is most important to wash thoroughly any aggregate containing clay or other foreign matter. Natural gravel containing both coarse and fine aggregate should be separated by washing it through $\frac{3}{16}$ -in. and $\frac{3}{4}$ -in. square screens (or $\frac{1}{8}$ -in. and $\frac{1}{2}$ -in. long-mesh screens) so that the proportion of coarse to fine may be definitely determined. In this way an aggregate of uniform grading can be fairly well ensured.

Concrete Proportions.

The proportions of the cement and aggregates used in making pre-cast piles should be adjusted so that the resulting concrete has a high compressive strength and impermeability, the latter being of even greater importance than strength because it is on this quality that the durability of the pile depends. The really important factors in securing successful results are a rich mixture, preferably not less than 1 : 1 $\frac{1}{2}$: 3, well-proportioned aggregate, and proper curing. Indeed, the concrete should be of the quality described as "High-grade" in the Recommendations for a Code of Practice for Reinforced Concrete.*

Fresh water is generally specified for gauging, and the proportion of water to each batch should be strictly controlled so as to maintain the consistency and strength of the concrete as uniform as possible. Slump tests should be made once or twice daily, since a small variation in the water content, and consequently the slump, causes a very large variation in the strength and permeability of the concrete. It should also be borne in mind that if the concrete is very dry much more careful ramming is required to prevent voids. For this reason it is not wise to adopt too small a slump—2 in. to 6 in. are useful limits—but whatever slump is adopted it should not be changed unless for some good reason. In warm weather and in the case of piles made with aluminous cement the slump should be increased a little. The slump test is described in Appendix VI.

Shape of Cross Section.

Reinforced concrete piles are usually square, rectangular, or octagonal in cross section. Sheet piles of tee section have also been used in some instances. Generally speaking, the lengths of the sides of square piles are between 10 in. and 24 in. Circular pre-cast reinforced concrete piles are seldom used as they possess no advantage over octagonal piles and the shuttering is not so simple.

Square and rectangular piles are best where considerable transverse strength is required and are therefore used in retaining walls where definite bending moments have to be resisted. Where high compressive strength is necessary octagonal piles—in which the vertical bars are generally bound by spirals instead of separate links—are sometimes preferred.

* "Report of the Reinforced Concrete Structures Committee of the Building Research Board with Recommendations for a Code of Practice for the Use of Reinforced Concrete in Buildings." (London : H.M. Stationery Office. Price 1s. 3d. net.)

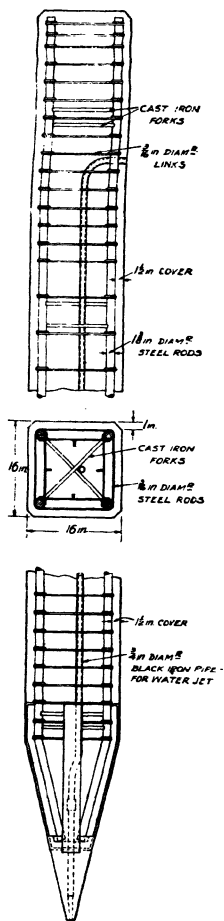


Fig. 1.—Typical Square Pile.

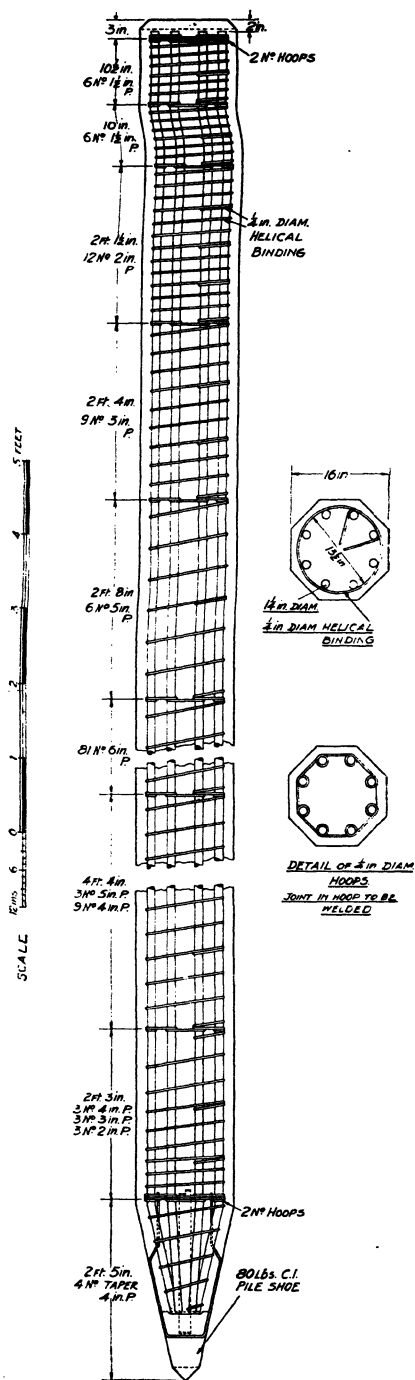


Fig. 2.—Typical Octagonal Pile.

Cover.

The thickness of concrete covering the main bars should never be less than $1\frac{1}{2}$ in. In cases where the steel is specially liable to corrosion the cover should be increased to 2 in. It should be remembered that reinforced concrete which is not wholly immersed in sea water, i.e. the portion above tide level, is peculiarly vulnerable in this respect, and good cover is here essential for long life. The disadvantage of a thick cover is the increase in weight and cost which is involved.

Reinforcement.

Figs. 1 and 2 show the reinforcement for a typical square and an octagonal pile. Reinforcement for square or rectangular piles generally consists of four

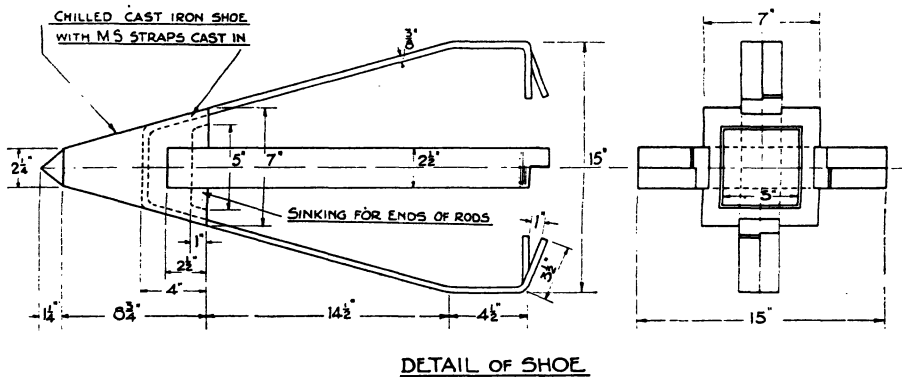


Fig. 3.

main longitudinal bars, one at each corner, extending throughout the length of the pile. Where considerable bending stresses occur due to lifting or to lateral pressure or to the pile having to act as a long column additional bars may have to be provided. Generally the bars range in diameter from 1 in. to 2 in.

The longitudinal bars are connected by a series of links which are usually $\frac{3}{16}$ in. to $\frac{1}{2}$ in. diameter and spaced at intervals of about 4 in. to 12 in. throughout the length of the pile, except at the head and foot where they are closer. The head and foot of a pile need to be specially strong because, as will be explained later, the stresses induced by driving are generally greatest at the two ends. It is specially important to strengthen the foot as damage here cannot be detected. The links are sometimes made double as in Fig. 1 connecting two adjoining bars, and sometimes single and passing round all four bars. In either case they should be bent on a rigid frame of bars truly spaced so as to fit tightly on the bars when in position.

At intervals of a few feet pairs of cast-iron or mild-steel struts, known as forks or spacers, are placed diagonally between opposite main bars to prevent them closing in while they are being surrounded with concrete and to ensure that the links fit tightly round the bars.

The reinforcement for octagonal piles consists generally of eight main longitudinal bars surrounded by continuous steel spirals of $\frac{1}{4}$ -in. to $\frac{3}{4}$ -in. diameter and 1-in. to 3-in. pitch. In this case also stiff spacers must be introduced to

ensure that the spirals are tight against the main bars. Also, as in the case of square piles, it is important to increase the lateral reinforcement at the head and foot.

Pile Shoes.

For pre-cast reinforced concrete piles of square cross section, the length of the point (that is, the portion from where the section reduces to the tip of the shoe) is generally from once to twice the diameter of the pile.

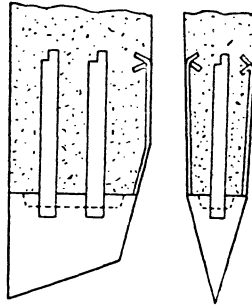


Fig. 4.—Detail of Shoe for Sheet Pile.

Experiments made by the late Mr. H. A. Reed during the Trafford Wharf reconstruction at Manchester to determine the effect of the angle of the shoe on the penetration obtained showed that the shape of the shoe had little



Fig. 5.—Rock Shoe.

effect on the driving resistance of the pile. Blunt points (60 deg.) were fitted to many of the piles and it was found that there was no appreciable difference between the results of driving these and piles with shoe-angles of 22 deg.

The favourite type of shoe has a square or octagonal point and is connected to the concrete by four mild steel straps. Turned-in straps give a good hold on the concrete, and it is not considered necessary to connect these by transverse bolts. It is, however, important that a part of each strap should extend beyond

the tapered portion of the pile for at least 4 in. exclusive of the length turned into the concrete.

The security of the point and shoe should be ensured by locating the bars in such a way that the concrete can be packed closely around them. For this reason the cast-iron point should be of such a length that its top is large enough for the bars to lie on it without overcrowding. A recess should be provided in the top of the point to prevent the bars spreading, and throughout the taper and just above it the links should be closely spaced.

A typical diamond-point for a 15-in. square pile is shown in *Fig. 3*. In this the overall length of the solid point is 10 in. and its section at the top is 7 in. square. The recess for the main bars is 5 in. square by 1 in. deep, and there are four straps of $2\frac{1}{2}$ -in. by $\frac{3}{8}$ -in. mild steel. These extend $4\frac{1}{2}$ in. along the main body of the pile and are split and turned in for a distance of $3\frac{1}{2}$ in.

Shoes for sheet piling (*Fig. 4*) are similar but are made with a chisel instead of a diamond-point and are sloped at about 1 in 4 to induce each pile to close up to its neighbour. Shoes may be omitted if the ground is very soft. Where piles are to be water-jetted, or driven a small distance into rock, shoes of special design (*Fig. 5*) are used. In the type illustrated a long pointed steel extension protrudes through the solid portion of a pile shoe of the usual shape.

CHAPTER II

DESIGN OF PILES

Strength.

PILES are commonly used in two ways, (1) as "bearing piles" acting as columns carrying the weight of a superstructure and of any loads imposed upon the latter, or (2) as "sheet piles" driven in a row to form a retaining wall capable of resisting the lateral pressure of earth on one side of it.

As regards bearing piles, in order safely to bear a permanent load of 50 tons, for instance, a pile must not only be strong enough to withstand the compressive stresses induced in it as a column by that load, but it must be driven to such a depth, or to such a stratum, that it will not sink under this load. Also, if precast, it must be designed to withstand the impact stresses induced during driving to this depth or stratum. As will be explained later, the latter stresses may be far greater than those induced by the permanent load.

Sheet piles used as a retaining wall must be strong enough to resist the bending stresses induced by the lateral pressure of the earth they support, and also to resist the vertical compression set up by the retained earth and by any superimposed load.

Furthermore, both classes of piles, if precast, must be strong enough to resist stresses due to bending while being lifted.

From these considerations it will be seen that a pile must be designed and handled in such a manner as to guard against damage in four ways:

(1) By crushing under the permanent load owing to lack of compressive strength as a column (short or long).

(2) By settlement under the permanent load owing to insufficient ground resistance.

(3) By crushing while being driven owing to lack of impact strength.

(4) By tension or compression set up by bending moments when acting as a retaining wall or while being lifted.

Bearing Piles.

In order to get a rough idea of what section of pile is likely to be required to carry a certain load it may be said that its cross-sectional area should be such that the load will impose a stress of something like $\frac{1}{8}$ ton per square inch. Thus the load borne by piles of various sizes may be roughly estimated as follows:

Scantling	Gross area (sq. in.)	Approximate permanent load (tons)
10 in. by 10 in.	100	20
12 in. by 12 in.	144	29
15 in. by 15 in.	225	45
18 in. by 18 in.	324	65
12 in. octagonal	119	24
15 in. " 	186	37
18 in. " 	268	54

Whether a pile will safely carry more or less than the load suggested in this table depends on circumstances, and to decide the point it is necessary to investigate the effect of the various possible causes of damage mentioned. The usual methods of so doing will now be dealt with.

Column Strength.

Square Piles.—The load which a pile will bear depends upon the areas of steel and concrete and the quality of the latter. The usual formula for the safe load on a square or rectangular pile regarded as a short column is

$$P = cA_e + tA \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (\text{I})$$

where $P =$ safe load (lb.)

c = permissible stress on concrete (lb. per square inch)

A_c = cross-sectional area of concrete (sq. in.)

t = permissible stress on steel (lb. per square inch)

A = cross-sectional area of steel (sq. in.).

Other units than pounds and inches may be used provided they are used consistently throughout.

The following values for c and t are given in the Recommendations for a Code of Practice :

For High-grade concrete :

Mix	<i>c</i> (lb. per square inch)
I : I : 2	1000
I : I $\frac{1}{2}$: 3	880
I : 2 : 4	760

For B.S.S. steel :

$t = 13,500$ lb. per square inch.

For high-yield point steel :

$t = 15,000$ lb. per square inch.

EXAMPLE.—A 15-in. square pile made of 1:1½:3 High-grade concrete is reinforced with four 1½-in. B.S.S. bars.

To find the safe load (as a short column).

$$P = cA_c + tA$$

In this case

$$c = 880 \text{ lb. per square inch}$$

$$A_c = 219 \text{ sq. in.}$$

$t = 13,500$ lb. per square inch

$$A = 5.94 \text{ sq. in.}$$

$$P = 880 \times 219 + 13,500 \times 5.94$$

$$= 274,000 \text{ lb. or } 122 \text{ tons.}$$

It will be noticed that this formula apparently takes no account of the links, but in fact it is based on the assumption that these are present in sufficient quantities.

Although the Recommendations for a Code of Practice does not deal specially with piles it gives some rules about links for columns which may be followed, bearing in mind that for reasons previously given the spacing of the links should

be closer at the head and foot of the pile. Briefly the Recommendations state that the volume of transverse reinforcement should not be less than 0.4 per cent. of the gross volume, and that the pitch of the links shall not exceed 12 in. or the length of the side of the column or twelve times the diameter of any longitudinal bar. Also the Recommendations require that where the pitch is the maximum permitted the diameter shall be at least one-quarter of that of the largest longitudinal bar. These rules are admirable, although it must be admitted that many piles have been successfully driven with rather lighter transverse reinforcement in the main body of the pile than is required by the Code.

The size and spacing of the links at the head and foot of the pile are most important and the notes on this subject given on page 119 are worthy of careful study. It is there suggested that for a length from the extremities of $2\frac{1}{2}$ to 3 times the external diameter of the pile the volume of lateral reinforcement should be not less than 1 per cent. of that of the gross volume, and that the size and spacing should be such as to provide ample facility for placing the concrete.

Octagonal Piles.—For estimating the safe load on an octagonal pile (*Fig. 6*) the same formula (1) is used, and also another which takes the spiral reinforcement into consideration. The safe load should be found by both formulæ and the higher value adopted.

The second formula is

$$P = cA_k + tA + 2t_bA_b \quad (2)$$

In this expression

P = safe load (lb.).

c = permissible direct stress for the concrete (lb. per square inch).

A_k = cross-sectional area of concrete in the core (sq. in.).

t = permissible stress for longitudinal steel in direct compression (lb. per square inch).

A = cross-sectional area of longitudinal steel (sq. in.).

t_b = permissible stress in tension in spiral steel (lb. per square inch).

A_b = equivalent area of spiral steel (volume of spiral per unit length of column) (sq. in.).

Other units than pounds and inches may be used if used consistently.

The following values for c (High-grade concrete), t , and t_b are given in the Recommendations for a Code of Practice:

Mix	c (lb. per square inch)
1 : 1 : 2	1000
1 : $1\frac{1}{2}$: 3	880
1 : 2 : 4	760

	t (lb. per square inch).	t_b (lb. per square inch)
Mild steel	13,500	13,500
Mild steel with yield point not less than 44,000 lb. per square inch	15,000	15,000

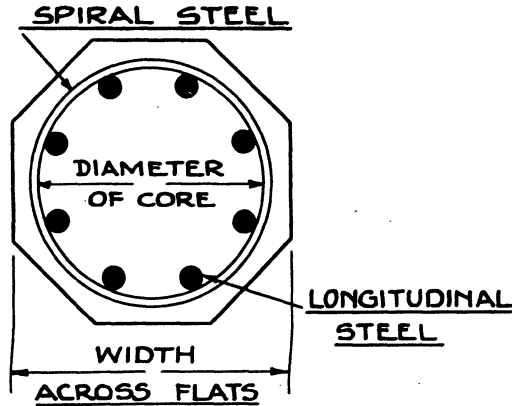


Fig. 6.—Cross Section of Octagonal Pile.

The Recommendations also require that the sum of the loads contributed by the concrete in the core and by the spiral ($cA_k + 2t_bA_b$) should not exceed $0.5uA_c$, where u is the concrete crushing strength from the 28-day works test—which should be not less than as follows for High-grade concrete:

Mix	u (lb. per square inch)
1 : 1 : 2	3750
1 : 1½ : 3	3300
1 : 2 : 4	2850

EXAMPLE.—An octagonal pile made of 1 : 1½ : 3 High-grade concrete is 16 in. across the flats and is reinforced with eight 1¼-in. mild steel bars and 7/16-in. mild steel spirals at 2-in. pitch. (Diameter of core = 13 in.) To find the safe load (as a short column).

From formula (1) $P = cA_c + tA$.

In this case $c = 880$ lb. per square inch.

$$A_c = 16^2 \times 0.8284 - 9.8 = 202 \text{ sq. in.}$$

$$t = 13,500 \text{ lb. per square inch.}$$

$$A = 8 \times 1.227 = 9.8 \text{ sq. in.}$$

$$\text{Hence } P = 880 \times 202 + 13,500 \times 9.8 \\ = 310,000 \text{ lb.} = 138 \text{ tons.}$$

Again from formula (2) $P = cA_k + tA + 2.0t_bA_b$

where $c = 880$ lb. per square inch

$$A_k = 13^2 \times 0.7854 - 9.8 = 123 \text{ sq. in.}$$

$$t = 13,500 \text{ lb. per square inch.}$$

$$A = 8 \times 1.227 = 9.8 \text{ sq. in.}$$

$$t_b = 13,500 \text{ lb. per square inch.}$$

$$A_b = \frac{3.141 \times 13.437 \times 0.1503}{2} \text{ cub. in. per inch long} \\ = 3.17 \text{ sq. in.}$$

$$\text{Hence } P = 880 \times 123 + 13,500 \times 9.8 + 2 \times 13,500 \times 3.17 \\ = 326,000 \text{ lb.} = 146 \text{ tons.}$$

Then the larger value of P (146 tons) may be regarded as the safe load.

Appendix I contains the safe short-column loads on various square and octagonal piles.

Long-Column Strength.

If a pile, when subject to its maximum load, has an unsupported length of more than fifteen diameters, its safe load as a short column must be multiplied by a coefficient to give its safe load as a "long" column. The coefficient can be ascertained from the following table.

TABLE 1.

$\frac{l}{d}$	15	18	21	24	27	30	33	36	39	42	45
Coefficient	1.0	0.9	0.8	0.7	0.6	0.5	0.4	0.3	0.2	0.1	0

In the table l = the effective length of the pile (in.)

d = the least lateral dimension of the pile (in.).

Resistance to Settlement.

The ultimate load which a driven pile will carry without further settlement may be estimated in two ways, (1) by a calculation based on the set produced by the hammer blow during the latter part of the driving, and (2) by a calculation based on the bearing power of the soil beneath the foot of the pile and of the frictional resistance of the soil around its sides. These may be called the dynamic and static methods respectively. The first method is the most usually employed and in our present state of knowledge the more reliable.

It seems obvious that the best way to estimate the bearing capacity of a driven pile is to observe the set produced by a hammer of known weight falling a known height, and many formulæ have been devised to give this result. Unfortunately most of them entirely neglect such important factors as the ratio of the weight of the hammer to that of the pile, the length, area, and modulus of elasticity of the pile, and the condition of the pile head which receives the blow. These factors are taken into consideration by Mr. A. Hiley, A.M.Inst.C.E., in a formula which he first published in 1922 and which is coming into general use.

It takes the form

$$R = \frac{Wh}{s + \frac{c}{2}} \times \eta \quad \dots \dots \dots (3)$$

where R = the ultimate resistance overcome in driving (tons)

W = weight of the hammer (tons)

h = height of free fall of the hammer (in.)

s = set or penetration of the pile per blow (in.)

c = temporary elastic compression of pile and cap (in.)

η = a coefficient of efficiency (see later).

Other units than tons and inches may be used for these symbols, provided that they are used consistently throughout.

To obtain accurate results the values of these symbols must be estimated with care.

The value of h should not necessarily be the total fall of a drop hammer (whose effect may be lessened by friction, etc.), but a "free fall" corresponding to the velocity of the hammer at the time of striking. If H is the total fall and h the free fall it may be taken that

$h = H$ for drop hammer released by monkey trigger.

$h = 0.8H$ for drop hammer actuated by friction winch.

$h = 0.9H$ for single-acting steam hammer.

If the hammer is a double-acting steam hammer it will be necessary to substitute for Wh

$$WS + SAM$$

where W = weight of hammer (moving ram only) (tons)

S = stroke (in.)

A = net area of piston acted on by steam (sq. in.)

M = mean steam pressure in cylinder (tons per square inch).

The symbol c represents the temporary compression of pile and helmet, etc. It is made up of three elements:

(1) The elastic compression of the pile, which we will call C_1 ;

(2) The temporary compression of the dolly and helmet, which we will call C_2 ;

(3) The temporary quake of the ground, which we will call C_3 .

c represents the sum of these, or

$$c = C_1 + C_2 + C_3.$$

Each of these three elements depends on (and is proportional to) the compressive stress produced in the pile by the resistance R . If A_p is the sectional area of the pile (sq. in.) it is evident that the compressive stress is $\frac{R}{A_p}$ tons per square inch.

Mr. Hiley gives the following values of C_1 , C_2 , and C_3 .

TABLE 2.
VALUE OF C_1 .

For reinforced concrete pile $0.0134 \frac{R}{A_p} \times L$ (inch)

For timber pile $0.018 \frac{R}{A_p} \times L$ (inch)

where L = length of pile in feet.

VALUE OF C_2 .

Reinforced concrete pile with pad, say, 1 in. thick but no dolly or helmet $0.11 \frac{R}{A_p}$ (inch)

Reinforced concrete pile with short dolly or helmet with, say, 4-in. packing $0.56 \frac{R}{A_p}$ (inch)

Timber pile $0.22 \frac{R}{A_p}$ (inch)

VALUE OF C_3 .

$$0.22 \frac{R}{A_p} \text{ (inch)}$$

As regards the coefficient η , its value as given by Mr. Hiley is set out in *Table 4*. It depends partly on the ratio of the weight of the pile (with helmet, dolly, etc.) to that of the hammer (moving parts). This is referred to in *Table 4* as $\frac{P}{W}$. It also depends on a coefficient referred to as e in *Table 4*, which depends on the nature of the hammer and of the head of the pile or helmet receiving the blow. The values of e as given by Mr. Hiley are set out in *Table 3*.

TABLE 3.

For steel ram of double-acting hammer striking on steel anvil and driving reinforced concrete pile	$e = 0.5$
For cast-iron ram of single-acting or drop hammer striking on head of reinforced concrete pile	$e = 0.4$
Single-acting or drop hammer striking a well-conditioned wood cap of helmet in driving reinforced concrete piles or directly on head of timber pile	$e = 0.25$
For a deteriorated condition of the head of pile or of dolly	$e = 0$

Having found the value of $\frac{P}{W}$ and of e , *Table 4* gives the values of η as given by Mr. Hiley.

TABLE 4.

Ratio of P/W	$e = 0.5$	$e = 0.4$	$e = 0.25$	$e = 0$
$\frac{1}{2}$	0.75	0.72	0.69	0.67
1	0.63	0.58	0.53	0.50
$1\frac{1}{2}$	0.55	0.50	0.44	0.40
2	0.50	0.44	0.37	0.33
$2\frac{1}{2}$	0.45	0.40	0.33	0.28
3	0.42	0.36	0.30	0.25
$3\frac{1}{2}$	0.39	0.33	0.27	0.22
4	0.36	0.31	0.25	0.20
5	0.31	0.27	0.21	0.16
6	0.27	0.24	0.19	0.14
7	0.24	0.21	0.17	0.12
8	0.22	0.20	0.15	0.11

There are various ways in which this formula can be of service. They can best be explained by examples.

It can be used in the following manner to ascertain what set is needed for a given hammer blow to develop any required resistance :

A pile is required to bear a permanent load of 45 tons, and, say, a test load during driving of 90 tons. It is proposed to make it 15 in. by 15 in. and 50 ft. long. A single-acting steam hammer weighing 3 tons and falling 3 ft. 6 in. will be used with a helmet and short dolly weighing 10 cwt. What set will be required ?

In this case in formula (3)
$$R = \frac{Wh}{s + \frac{c}{2}} \times \eta.$$

$$\begin{aligned} R &= \text{resistance (tons)} \\ W &= 3 \text{ tons} \\ h &= 0.9 \times 42 = 38 \text{ in.} \\ s &= \text{set (in.)} \\ c &= C_1 + C_2 + C_3. \end{aligned}$$

And from Table 2

$$C_1 = 0.0134 \times \frac{R}{A_p} \times 50 = 0.67 \frac{R}{A_p}$$

$$C_2 = 0.56 \times \frac{R}{A_p} = 0.56 \frac{R}{A_p}$$

$$C_3 = 0.22 \times \frac{R}{A_p} = 0.22 \frac{R}{A_p}$$

$$\text{Hence } c \quad k = 1.45 \frac{R}{A_p}$$

$$\text{or, since } A_p = 15 \text{ in.} \times 15 \text{ in.,}$$

P = weight of pile

$$\begin{aligned} &= 50 \times \frac{15 \times 15}{144} \times \frac{150}{2240} = 5.21 \\ &\quad + \text{weight of helmet} \quad 0.50 \end{aligned}$$

$$= 0.0064R$$

$$= 5.71 \text{ tons}$$

$$\frac{P}{W} = \frac{5.71}{3.0}$$

$$= 1.90$$

e (from Table 3)

$$= 0.25.$$

Hence from Table 4,

$$\eta = 0.38.$$

$$\text{Hence from formula (3) } R = \frac{3 \times 38}{s + \frac{0.0064R}{2}} \times 0.38$$

$$= \frac{43.32}{s + 0.0032R}$$

or

$$s = \frac{43.32}{R} - 0.0032R.$$

If then the desired resistance R be 90 tons

$$\begin{aligned} s &= \frac{43.32}{90} - 0.0032 \times 90 \\ &= 0.48 - 0.29 = 0.19 \text{ in.} \end{aligned}$$

or, say, 5 blows to the inch.

The formula can of course also be used to find the ground resistance for a given set—that is, given c to find R . In this form the equation becomes a quadratic and is somewhat tedious to solve. It is quite simple, however, to find the value of R for $s = 0$ and then the value of s for two or three other values of R , from which other desired relations between R and s can be interpolated.

The formula can also be used to assist in the selection of a suitable hammer. For instance, if it were proposed to use a 1-ton drop hammer operated by a friction winch and falling 10 ft. 6 in. instead of a 3-ton steam hammer falling 3 ft. 6 in., the formula would show that the driving effect would be very different. In this case some of the symbols in the formula would be varied.

$$\begin{aligned}
 R &= \text{resistance (tons)} \\
 W &= 1.0 \text{ ton} \\
 h &= 0.8 \times 126 = 101 \text{ in.} \\
 s &= \text{set (in.)} \\
 c &= (\text{as before}) = 0.0064 R \text{ (in.)} \\
 P &= (\text{as before}) = 5.71 \text{ tons} \\
 \frac{P}{W} &= \frac{5.71}{1.0} = 5.71 \\
 e &= (\text{from Table 3}) = 0.25.
 \end{aligned}$$

$$\text{Hence from Table 4,} \quad \eta = 0.20.$$

$$\text{Hence from formula (3)} \quad R = \frac{1.0 \times 101}{s + 0.0032 R} \times 0.20.$$

By inserting $s = 0$ we can find what the value of R will be when the pile is driven to refusal.

$$\begin{aligned}
 \text{Here } R &= \frac{1.0 \times 101}{0 + 0.0032 R} \times 0.20 \\
 \text{or } R^2 &= 6300 \\
 \text{or } R &= 79 \text{ tons.}
 \end{aligned}$$

In other words we see that even if it is driven to refusal a 1-ton hammer falling 10 ft. 6 in. will not suffice to develop the desired resistance of 90 tons, thus illustrating the inefficiency of a light hammer in driving a heavy pile.

The formula is useful in yet another way. In unexplored ground it is not always easy to estimate what length of pile will be required. To solve this problem it is often best to drive a few trial piles which may be short timber piles (say 20 ft.) driven with a comparatively light hammer (say 1 ton) on a low frame. The short pile may not be long enough but it can be quickly lengthened, the new length being joined to the first by a dowel and a pair of bolted fish plates. The set of the pile and the fall of the hammer should be noted every few feet in depth, and the resistance of the ground at those levels can then be calculated by using the formula with the appropriate constants.

The specification published by the Institution of Structural Engineers advocates a test pile consisting of a steel tube which can be driven into the ground and so arranged that the point can be driven down independently of the tube. By this means it is possible to determine how much of the ground resistance is due to the sides of the pile (friction) and how much to the end (displacement of earth).

As already stated the observation of the set under a given hammer blow is the most satisfactory way of estimating the bearing power of a pile. In most cases the pile becomes even safer as time goes on, because the friction (or adhesion) between the sides of the pile and the surrounding ground, which is

little or nothing while the pile is being driven, increases as the material closes in around the pile, especially if the material is clay.

That the factor of safety of a pile increases with time is a rule with exceptions. For instance, cases have occurred where piles have been driven through "made" ground which in the process of settling down has dragged the piles down with it.

Usually, however, the friction which develops between the sides of the pile and the surrounding ground is an advantage. Indeed in certain cases it is almost the only support that can be relied upon. In some cases piles have been easily driven into soft mud of such a depth that no firm stratum was reached. None the less after a time the piles have been capable of bearing a considerable load owing to skin friction. Formulæ have been devised for such cases, but probably the only satisfactory way to estimate the bearing power of such piles would be by an actual load test.

Tapered Piles.

Although certain types of piles which are cast in situ are made in tapered form, pre-cast piles are invariably moulded of the same section throughout. This is partly no doubt because a tapered pile would be weak to resist the bending moments set up during lifting, and partly because there seems to be no advantage as regards bearing power in such a pile. The friction between the sides of the pile and the surrounding earth would no doubt be greater during driving with a tapered pile than with a parallel pile, but eventually there should be little to choose between the two in this respect, and the bearing resistance at the foot of the parallel pile would be greater.

Stresses during Driving.

As mentioned previously, the compressive stress imposed on a pile by the permanent load which it is designed to carry is often greatly exceeded by the stress imposed during driving, so much so that the pile head is sometimes damaged in the process. This is a matter of minor importance, as the pile head almost invariably has to be cut away and re-concreted when joining it to the cap or beam above it. But it has happened on occasions that a pile which has been subjected to prolonged hard driving into an exceptionally firm stratum has been crippled below ground. This is a serious matter, as the pile is thus injured in a vital part and it is not always easy to detect its occurrence. A pile so treated may appear to have been sinking slowly and steadily under the hammer blows, and yet in reality its foot may have been no longer penetrating the hard ground, and the pile may have been merely sinking because it was being crushed at the foot.

If, therefore, a pile is to be subjected to prolonged hard driving special precautions should be taken to prevent damage. It is obviously very desirable in such a case to estimate if possible the stresses produced by the hammer blow, and to limit the force of the blow in such a way that the stresses produced shall not exceed the impact strength of the concrete.

Estimating the stresses produced in a pile by a hammer blow is not a very simple matter. Mr. Hiley in his paper "Pile-driving Calculations" ("Struc-

tural Engineer," July and August 1933) states that the force on the pile head will not be less than $R \times \frac{1}{\sqrt{\eta}}$

where R and η have the values assigned to them on page 12. Applying this formula to the example given on page 14

where

$$R = 90 \text{ tons}$$

$$\eta = 0.38$$

it will be seen that the forces on the pile head will be not less than

$$90 \times \frac{1}{\sqrt{0.38}} = 146 \text{ tons,}$$

or at least 50 per cent. more than the ground resistance.

Stresses during driving have been the subject of an investigation by the Building Research Station of the Department of Scientific and Industrial Research. Their results have been published in a paper by Messrs. Glanville, Grime and Davies in the "Journal of the Institution of Civil Engineers" (December 1935), and a summary of the research has been published in a subsequent number of the same journal (April 1936) under the title of "Notes and Practical Suggestions on Pile Driving." By permission of the Institution of Civil Engineers these notes are given in Appendix IV.

They state that the compressive stress produced by the hammer blow passes down the pile as a wave to its foot, where it is reflected as a compression or a tension according to whether the ground resistance at the foot is great or small. Unless this ground resistance is very great or the pile very short, the maximum stress is at the head of the pile. The head stresses depend upon the weight and drop of the hammer and also upon the ratio of the weight of the hammer to the weight per foot run of the pile. They also depend largely on the stiffness of the cushion (i.e. the dolly and the helmet packing) at the pile head. The stiffness of helmet packings in ordinary use tends to increase as hammering proceeds.

Curves deduced from the investigations are given in the notes (see *Figs. 80-82*), from which, if the weight of the hammer and the weight per foot run of the pile and the stiffness of the cushion are known, the maximum drop that should be given to the hammer in order to avoid exceeding certain limits of head stress can be read off. A device was tested by which the actual head stresses during driving could be ascertained, but it is said that this needs further experience.

The stresses at the foot of the pile, as stated previously, are generally less than those at the head unless the ground resistance is very great. They depend on this ground resistance, and also on the ratio of weight of hammer to weight per foot run of pile, and also on the stiffness of the cushion. Curves are given in *Fig. 82* from which, if the weight of hammer and weight per foot run of pile and the stiffness of the cushion are known, the minimum "equivalent elastic set" which can be given to the pile without exceeding certain defined limits of stress can be read off. The "equivalent elastic set" is defined as twice the plastic (or permanent) set plus the elastic (or temporary) set. A device for measuring this set by attaching it to any pile while being driven is described in the notes.

The notes also emphasise certain precautions in the matter of materials and workmanship to be taken in the case of piles which have to be subjected to hard driving, and therefore to high stresses. Particularly the concrete should be rich and well proportioned, and well reinforced at the head and foot. The pile head should be truly formed and the packing evenly distributed. The hammer should be central and its travel in line with the pile.

The results obtained from this investigation suggest that the stresses at the pile head during hard driving often approach, or perhaps exceed, the ultimate strength of the concrete. For reasons previously given this is, generally speaking, unimportant. But it is good practice to limit the hammer blow to something less than the test blow until the pile is nearing its final depth, and to cease driving when it is reasonably certain that the test has been attained.

Bending Moments Caused by Lifting Piles.

The lifting of piles for stacking and for driving is a matter that calls for careful consideration, as high bending stresses are set up in the pile during the process which if not provided for may cause serious damage.

It is often required to lift a pile for stacking when it is perhaps only a week old, and when its concrete strength may be only half to three-quarters of that which it will attain after a month. In order to minimise the stresses in the pile it is usual to lift for stacking at two points. With long and heavy piles it may be necessary to use a strong steel joist as a spreader and suspend the pile from it at three or four points. It is necessary therefore to ascertain where

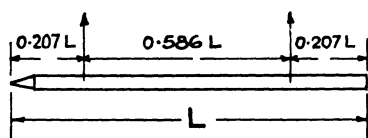


Fig. 7.—Two-point Suspension.

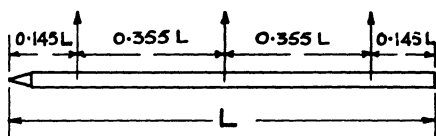


Fig. 8.—Three-point Suspension.

these lifting points should be so that the bending moments should be as small as possible, to estimate the amount of the maximum bending moment thus obtained, and to ascertain the stresses induced in the concrete and steel by such bending moments to ensure that these do not exceed safe limits.

The calculations required for the best positions of points of suspension and the corresponding bending moments are fairly simple if the pile be assumed to be uniform in weight, strength, and stiffness throughout. The results are as follows.

The diagrams (Figs. 7 and 8) give the best positions for points of suspension for two-point and three-point suspension.

The maximum bending moments produced by such suspension is given by the following formulæ :

$$\text{Two point, } M = \frac{WL}{46.63} \text{ in.-lb.}$$

$$\text{Three point, } M = \frac{WL}{95} \text{ " "}$$

REINFORCED CONCRETE PILING

where M = bending moment (in.-lb.)
 W = weight of pile (lb.)
 L = length of pile (in.)

EXAMPLE.—To find (1) the best positions for the supports when lifting a 15-in. square reinforced concrete pile 55 ft. long at two points, and (2) the maximum bending moment on the pile.

In this case $L = 55 \text{ ft.} = 660 \text{ in.}$

$$W = 55 \times \frac{15 \times 15}{144} \times 144 = 12,375 \text{ lb.}$$

The distances of the supports from the ends of the pile are $0.207L = 136.6 \text{ in.}$, say, 11 ft. 5 in.

The bending moment is

$$M = \frac{12,375 \times 660}{46.63} = 175,200 \text{ in.-lb.}$$

When the pile is to be hoisted for driving it is most convenient to suspend it at one point near the head. By this time the pile must have attained something like its final strength to stand the driving, but it is necessary again to ascertain the bending moment and the stresses produced by such suspension so as to make sure that the latter are not excessive.

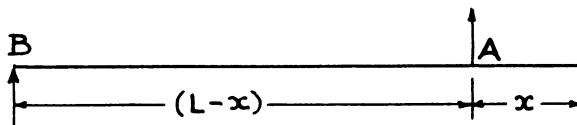


Fig. 9.—Pile Suspended near the Head.

In this case the bending moments set up are given by the formulæ (see Fig. 9)

$$M_A = \frac{Wx^2}{2L}$$

$$M_B = \frac{W}{8L} \left\{ \frac{L(L - 2x)}{L - x} \right\}^2$$

where

M_A = bending moment at A (in.-lb.)

M_B = maximum bending moment between A and B (in.-lb.)

W = weight of pile (lb.)

L = length of pile (in.)

x = distance of point of suspension from head (in.).

These bending moments are least (and alike) when

$$x = 0.293L, \text{ and in this case}$$

$$M_A = M_B = \frac{WL}{23.4} \text{ (in.-lb.).}$$

Other units than pounds and inches may be used if they are used consistently.

A pile can be lifted for driving at two points if a crane is available to hold the pile near the shoe while the pile-driving winch holds it near the head and hoists it.

Stresses Caused by Lifting : Square and Rectangular Piles.

Having ascertained the maximum bending moment on a pile caused by lifting (or by any lateral force) it is then necessary to determine whether the pile as designed is capable of resisting the stresses caused by the bending moment. It is usually sufficient for this purpose to ascertain the compressive stress on the outer face of the concrete and the tensile stress on the steel below the opposite face. These stresses should not exceed the safe limits which are given below.

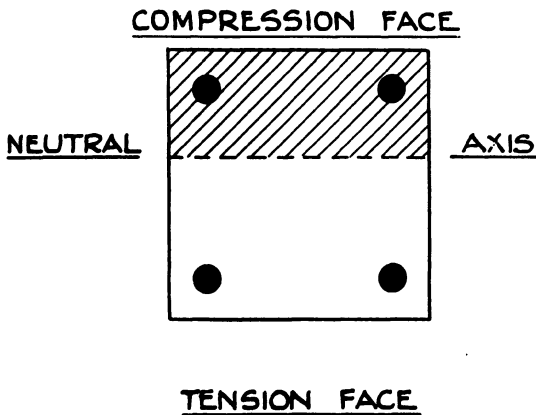


Fig. 10.

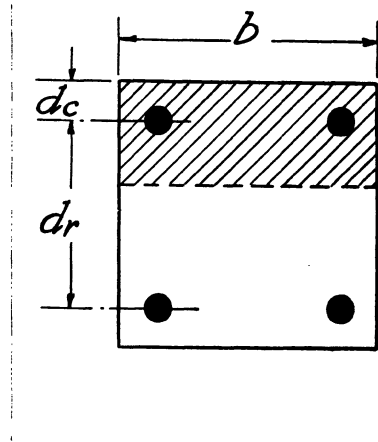


Fig. 11.

In this case we are dealing with a beam whose reinforcement at top and bottom is equal in area and symmetrically placed (Fig. 10).

It is usually assumed that the compressive stress is a maximum at the top edge of the pile and diminishes to nil at the neutral axis. It is also assumed that the steel in the compression zone shares the compressive stress and that for this purpose it is equivalent to concrete of m times its area (where $m = 15$), and that in the tension zone the tensile stress is taken by the steel alone.

The calculations are somewhat complicated and can be greatly simplified by further assuming that the centre line of resistance of the concrete in compression coincides with that of the compression steel. This is not far from being the case with piles of average size. Then the stresses induced by the bending moment may be expressed by the formulæ (Fig. 11)

$$c = \frac{M}{\left(\frac{b \times 3d_c}{2} + 9.33A_s \right) \times d_r} \quad \dots \dots \dots (4)$$

$$t = \frac{M}{A_s \times d_r} \quad \dots \dots \dots (5)$$

where c = maximum compressive stress in concrete (lb. per square inch)

t = tensile stress in tension steel (lb. per square inch)

M = bending moment (in.-lb.)

b = breadth (in.)

d_c = depth of centre of compression steel below compression face (in.)

A_s = area of tension steel (sq. in.)

d_r = depth of tension steel below compression steel (in.)

Other units than pounds and inches may be used if used consistently.

EXAMPLE.—A 15-in. by 15-in. pile with four $1\frac{3}{8}$ -in. bars and $1\frac{1}{2}$ -in. cover is subjected during lifting to a bending moment of 300,600 in.-lb. To find the stresses produced in the concrete and the steel:

In this case

$$b = 15 \text{ in.}$$

$$d_c = 2.19 \text{ in.}$$

$$A_s = 2.97 \text{ sq. in.}$$

$$d_r = 10.625 \text{ in.}$$

$$c = \frac{M}{\left(\frac{b \times 3d_c}{2} + 9.33A_s\right) \times d_r}$$

$$= \frac{300,600}{\left(\frac{45}{2} \times 2.19 + 9.33 \times 2.97\right) \times 10.625} = 368 \text{ lb. per square inch.}$$

$$t = \frac{M}{A_s \times d_r} = \frac{300,600}{2.97 \times 10.625} = 9526 \text{ lb. per square inch.}$$

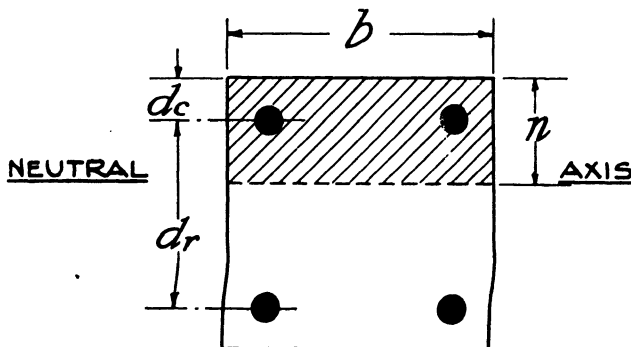


Fig. 12.

A more orthodox (and doubtless more correct) method of ascertaining the stresses caused by a given bending moment is as follows:

First find the depth of the neutral axis (Fig. 12) below the compression face. This is given by the formula

$$n = -\frac{29A}{b} + \sqrt{\left(\frac{29A}{b}\right)^2 + \frac{2A}{b}(15d_r + 29d_c)} \quad . \quad . \quad . \quad (6)$$

where n = the depth of the neutral axis below the compression face (in.)
 A = the area of steel in compression or of steel in tension (sq. in.)
 b = the breadth of the pile (in.)
 d_c = the depth of the centre of the compression steel below the compression face (in.)
 d_r = the depth of the centre of the tension steel below the compression steel (in.)

Next find the moment of inertia of the section. This is given by the formula

$$I = \frac{bn^3}{3} + 14A(n - d_c)^2 + 15A(d_r + d_c - n)^2 \quad . \quad . \quad . \quad (7)$$

where I = the moment of inertia of the section (inch units) and the other symbols are as previously stated.

Then the maximum compressive stress in the concrete is given by the formula

$$c = \frac{Mn}{I} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (8)$$

where c = the maximum compressive stress (lb. per square inch)

M = the bending moment due to lifting, etc., found as previously explained (in.-lb. units)

n = the depth of the neutral axis (as before) (in.)

I = the moment of inertia of the section (as before) (inch units)

and the tensile stress in the steel is given by the formula

$$t = \frac{15M(d_c + d_r - n)}{I} \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad . \quad (9)$$

where t = tensile stress in tension steel (lb. per square inch) and the other symbols are as previously stated.

Other units than pounds and inches may be used in these formulæ provided that they are consistently used throughout.

The compressive stress in the concrete should not exceed the following limits :

High-grade concrete,	1 : 1 : 2,	1250 lb. per square inch				
" "	" 1 : 1½ : 3,	1100 "	"	"	"	"
" "	" 1 : 2 : 4,	950 "	"	"	"	"

The tensile stress in the steel should not exceed

B.S.S. steel : 18,000 lb. per square inch.

High-yield-point steel : 20,000 lb. per square inch.

EXAMPLE.—Take the previous example in which a 15-in. square pile reinforced with four 1½-in. bars is subjected during lifting to a bending moment of 300,600 in.-lb., and find the stresses produced in the concrete and in the steel.

First find the depth of the neutral axis from formula (6)

$$n = -\frac{29A}{b} + \sqrt{\left(\frac{29A}{b}\right)^2 + \frac{2A}{b}(15d_r + 29d_c)}.$$

In this case $A = \text{area of steel in tension} = 2.97 \text{ sq. in.}$
 $b = 15 \text{ in.}$
 $d_c = 2\frac{3}{16} \text{ in.} = 2.19 \text{ in.}$
 $d_r = 10\frac{5}{8} \text{ in.} = 10.625 \text{ in.}$

Hence $n = -5.74 + \sqrt{32.98 + 88.23} = 5.27 \text{ in.}$

Then find the moment of inertia from formula (7)

$$I = \frac{bn^3}{3} + 14A(n - d_c)^2 + 15A(d_r + d_c - n)^2$$

In this case $b = 15 \text{ in.}$
 $n = 5.27 \text{ in.}$
 $A = 2.97 \text{ sq. in.}$
 $d_c = 2.19 \text{ in.}$
 $d_r = 10.625 \text{ in.}$

$$\therefore I = \frac{15 \times (5.27)^3}{3} + [14 \times 2.97 \times (5.27 - 2.19)^2] + [15 \times 2.97 \times (10.625 - 5.27)^2]$$

$$= 3660 \text{ in.}^4$$

Then find the maximum compressive stress in the concrete from formula (8)

$$c = \frac{Mn}{I} = \frac{300,600 \times 5.27}{3660} = 430 \text{ lb. per square inch.}$$

Then find the maximum tensile stress in the steel from formula (9)

$$t = \frac{15M(d_c + d_r - n)}{I} = \frac{15 \times 300,600(10.625 - 5.27)}{3660} = 9300 \text{ lb. per sq. inch.}$$

If the concrete of which the pile is composed is High-grade in the proportions of 1 : 1½ : 3, which at a month old should safely bear 1100 lb. per square inch compressive stress, it will probably not be overstressed by 430 lb. per square inch at a week old. The steel is obviously not overstressed.

Stresses Caused by Lifting : Octagonal Piles.

IN order to ascertain whether an octagonal pile is strong enough to resist the stresses induced by a given bending moment we must, as in the case of a rectangular pile, first find the depth of the neutral axis below the compression edge and then the moment of inertia of the section. This is not quite so simple as in the case of a square or rectangular pile, and indeed can most easily be done by a graphic method which may be described as follows.

Fig. 13 shows the cross section of an octagonal pile reinforced with eight bars. The upper part of the section is in compression and the steel in the lower part is in tension. The concrete in the tension area is, as usual, ignored.

Divide the cross section into convenient small strips by horizontal lines

marked 1, 2, 3, 4, 5 in the compression area and calculate the area of each strip, adding in each case fourteen times the area of the steel bars where they occur.

Form the compression vector polygon by drawing any vertical line $P^1-0-10-P^2$ and setting off on the line of the top edge of the section produced, and to any convenient scale, the lengths $0-1$, $1-2$, $2-3$, etc., equal to the areas of the strips between the lines 0, 1, 2, 3, etc. Choose a pole P^1 at any convenient distance above 0, and join P^11 , P^12 , P^13 , etc.

Now form the tension vector polygon by setting off on the line of the lower tension bars produced, and to the same scale as is used for the compression vector polygon, the lengths $10-11$, $11-12$ equal to fifteen times the area of the two lower

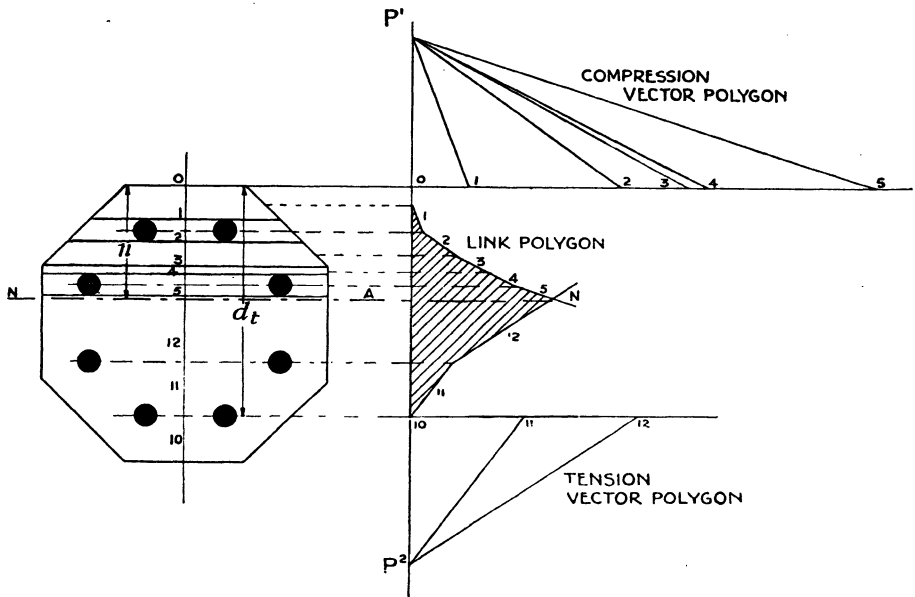


Fig. 13.—Graphical Construction for the Moment of Inertia of an Octagonal Pile.

and the two upper tension bars respectively. Choose the pole P^2 so that $P^210 = P^10$ and join P^211 and P^212 .

Now form the link polygon by projecting horizontal lines from the centre of gravity of each compression strip and from each pair of tension bars on to the vertical line $P^1-0-10-P^2$. Through the point opposite the centre of gravity of compression strip $0-1$ draw the line marked 1 parallel to P^11 , and similarly draw the lines 2, 3, 4, and 5 parallel to P^12 , P^13 , P^14 , and P^15 respectively. Also through the point opposite the lower tension bars draw the line marked 11 parallel to P^211 , and similarly draw line 12 parallel to P^212 and produce it to intersect line 5 in N . The horizontal line drawn through N to intersect the cross section will represent the position of the neutral axis and its depth n can now be ascertained by scaling.

Moreover, the area of the link polygon (as hatched) represents to scale the moment of inertia of the cross section of the pile, its actual value being determined as follows :

If the cross section be plotted to a scale of A inches to the inch
 and the areas on the vector polygons to a
 scale of B square inches to the inch
 and the actual area of the link polygon is C sq. in.
 and the polar distances of P^1 and of P^2 are each D in.
 then the moment of inertia

$$I = 2 A^2 \times B \times C \times D.$$

Having found the value of the moment of inertia the maximum compressive stress in the concrete can be found as before from the formula

$$c = \frac{Mn}{I}$$

where c is the maximum compressive stress on the concrete (lb. per square inch)
 M is the bending moment (in.-lb.)
 I is the moment of inertia (inch units)
 n is the depth of the neutral axis below the compression edge (in.),
 and the maximum tension in the steel can be found from the formula

$$t = \frac{15M (d_t - n)}{I}$$

where t = maximum tensile stress in steel (lb. per square inch)
 M = bending moment (in.-lb.)
 I = moment of inertia (inch units)
 d_t = depth of lower tensile steel bars from compression edge (in.)
 n = depth of neutral axis from compression edge (in.).

The compressive and tensile stresses thus found can be then compared with the permissible stresses for the concrete and steel employed.

The stresses caused by lifting are obviously greatest when the pile is lifted by one sling at or near its head while its toe rests on the ground. This is often done when pitching the pile for driving and then the pile is in effect supported at its extreme ends and the bending moment is $\frac{WL}{8}$.

Appendix I gives the safe moment of resistance of various square and octagonal piles.

Appendix II gives the maximum length of certain square and octagonal piles made with strong concrete which can be supported at the extreme ends without overstressing. If the piles are supported at the two "best" points ($0.207L$ from each end) these lengths can be rather more than doubled. But with piles of such lengths the question of stresses induced by lifting should be carefully studied, having regard to the strength of the concrete at the time of lifting and bearing in mind that any jerking of the crane and any inclination of the slings may increase the stresses to some extent.

In the case of certain piles of unusual section, such as T-shape piles, it may

be necessary to investigate the shearing and bond stresses and to arrange that these shall not be excessive. In the case of rectangular and octagonal piles it is usually sufficient to provide for the longitudinal tension and compression stresses. In any case the points at which the pile is to be slung should be determined before moulding and marked on it, preferably by inserting tubes through which lifting bolts can be passed.

Effect of Age on Strength of Concrete.

It will be realised that the permissible stresses given in the Recommendations for a Code of Practice are based on the concrete being a month old. It does not follow, of course, that a concrete pile must attain this age before being handled, or even before being driven. Some figures as to practice in this respect are given on page 68. But if the concrete is much less than 28 days old the deficiency in strength must be allowed for.

The growth of strength of good typical concretes at normal temperatures is well illustrated by the curves given in Appendix VII, page 123, which are based on a large number of experiments carried out at the Building Research Station. These, however, are laboratory results. In the field, and especially in cold weather, the rate of hardening may be slower. But the curves suggest that normal Portland cement concrete at ordinary temperatures attains more than half its 28-day strength in a week.

CHAPTER III

GROUPS OF BEARING PILES

WHEN the load on a column is too great to be carried by a single pile a group of two or more piles has to be provided. The spacing of such a group is a matter of importance. As already stated, bearing piles may be driven to a firm substratum or may be supported by means of friction by the earth which they penetrate. Indeed it may be said that most bearing piles are supported in both ways. But in any case it is evident that, unless the substratum is extraordinarily firm, two piles driven close alongside each other will not safely bear double the load that one of them will bear. In other words, groups of piles must be driven a sufficient distance apart. The subject is one which calls for further research, but meanwhile it may be said that it is customary to drive groups of piles at least three diameters apart centre to centre.

In cases where the group contains a large number of piles, it is considered good practice to arrange them in such a way that the load on each one is about the same, and if possible to drive all piles in the group to about the same level.

It should be borne in mind that when piles are grouped the soil may be affected to a considerable distance below the piles. It is important therefore that, in the case of heavy works, trial borings or deep trial piles should be put down to ascertain if there are any weak strata below the proposed level of the pile foot. It must also be recognised that in some cases clays are weakened by driving piles through them.

File Caps.

The design of a short pile cap calls for some consideration. It should be of ample dimensions as regards length and width to allow for the piles getting out of their intended position while being driven. A column can, of course, rest on one, two, or more piles. If it rests on one only and is not concentric with it, bending stresses are set up which must be provided for. Similarly a column on two piles, but not on the line joining the centres of the two piles, sets up bending stresses in the cap and piles and torsion stresses in the cap. If the eccentricity is considerable it may be necessary to drive an additional pile.

Otherwise the design of a single pile cap is a simple matter, as if both column and pile are correctly placed it is subject to compressive stress only. It should be deep enough to admit of the necessary bar overlap, which the Recommendations for a Code of Practice lay down as twenty-four times the diameter of the column bars. Unless the column is massive the usual transverse reinforcement links should be provided.

The design of a two-pile cap is not quite so simple. It might be thought that it is a beam with a point load and should be designed as such. But consider the combination shown in *Fig. 14* which represents a 14-in. by 14-in. column weighing (with its load) 80 tons, resting on a cap which distributes the load on to two 15-in. by 15-in. piles driven 3 ft. 9 in. apart centre to centre. A simple investigation will show that, although the tension and compression stresses induced by bending at the centre can be ascertained and provided for in the usual way, if the shear stresses be calculated as usual and an attempt be made

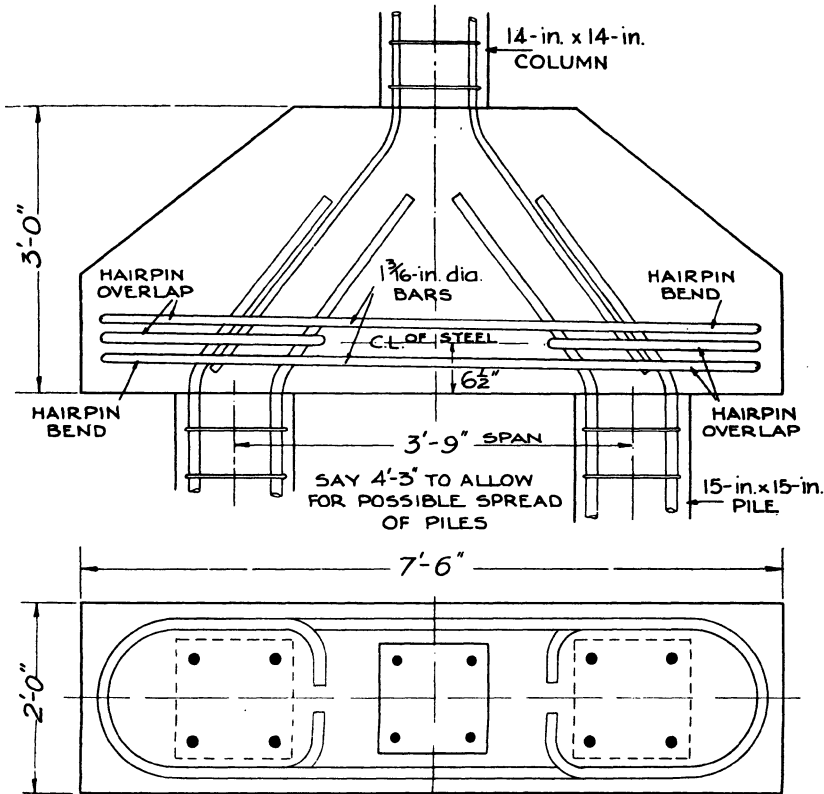


Fig. 14.—Two-pile Cap.

to provide against them by means of bent-up bars or stirrups, such a large amount of steel is needed that it is almost impossible to place it. In a case of this sort where the span is small, and the point load and consequently the total shear are comparatively great, it is more convenient and probably more correct to treat the cap as a truss rather than as a beam, and to regard the load as being transmitted from the centre to the supports by inclined compression in the concrete, the feet of the two concrete raking struts being prevented from spreading by the horizontal tension reinforcement. Briefly the application of this method of treatment to this particular case would be as follows:

First find the depth of the cap and the area of tension steel required. We

will assume that the concrete is to be a 1 : 2 : 4. Ordinary-grade mix with a safe stress in compression (due to bending) of 750 lb. per square inch and that the steel has a safe stress of 18,000 lb. per square inch. For this the ordinary formulæ applicable to a reinforced concrete rectangular beam may be used thus (Fig. 15) :

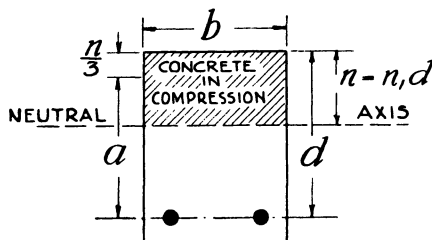


Fig. 15.—Cross Section of Pile Cap.

To find the depth

$$d = \sqrt{\frac{6M}{cbn_1(3 - n_1)}}$$

where d = effective depth (in.)

M = bending moment (2,005,000 in.-lb.) (see later)

c = maximum stress in concrete (750 lb. per square inch)

b = breadth = say 17½ in.

(Note—The actual width of beam adopted is 24 in. but for purposes of calculation it would seem prudent to limit the breadth to, say, 1½ times the width of the column (14.0 in.)

n_1 = ratio of neutral axis depth to total effective depth = 0.428 (see later).

$$\text{Hence } d = \sqrt{\frac{6 \times 2,005,000}{750 \times 17.5 \times 0.428 (3 - 0.428)}}$$

$$= 28.8 \text{ in.}$$

Note. M in this case is given by the formula (Fig. 16)

$$M = \frac{W(2L - l)}{8} + \frac{W'l}{8}$$

where M = bending moment (in.-lb.)

W = load on column (80 tons, say, 180,000 lb.)

W' = weight of cap (estimated as 4,000 lb.)

L = span—in this case 3 ft. 9 in. or 45 in., but, say, 51 in. to allow for possible spread when driving piles.

l = loaded length = 14 in.

$$\text{Hence } M = \frac{180,000(2 \times 51 - 14)}{8} = 1,980,000 \text{ in.-lb.}$$

$$+ \frac{4000 \times 51}{8} = 25,000$$

$$2,005,000$$

And n_1 is given by the formula

$$n_1 = \frac{mc}{t + mc}$$

where $n_1 = \frac{n}{d}$ = ratio of neutral axis depth to total effective depth.

m = modular ratio = 18.

c = maximum permissible compressive stress in concrete = 750 lb. per square inch.

t = maximum permissible tensile stress in steel = 18,000 lb. per square inch.

$$\text{Hence } n_1 = \frac{18 \times 750}{18000 + (18 \times 750)} = 0.428.$$

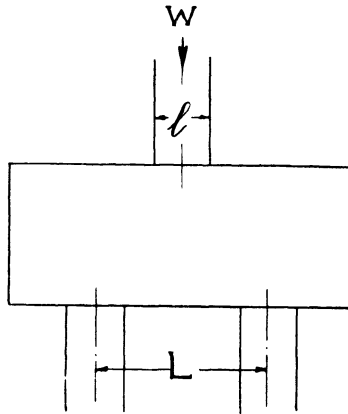


Fig. 16.—Elevation of Pile Cap.

The total depth must of course be greater than the effective depth. For reasons given later, for this type of truss the depth should be at least $d + \frac{n}{3}$ (in this case $28.8 + \frac{12.3}{3} = 32.9$ in.). In the finished design for other reasons it has been made 3 ft.

Next find the area of steel required. This is given by

$$A = \frac{cn_1bd}{2t}$$

where A = area of steel (sq. in.) and the other symbols are as previously given.

$$\text{Hence } A = \frac{750 \times 0.428}{2 \times 18,000} \times 17.5 \times 28.8 = 4.50 \text{ sq. in.}$$

Four $1\frac{3}{8}$ -in. bars will give this area with sufficient accuracy.

As the compression in the concrete is steeply inclined it is not necessary to provide reinforcement in the shape of stirrups or bent-up bars to resist shear

(or more accurately to resist diagonal tension) provided that two conditions are observed :

(1) The ties, in which the tensile stress is considered to be uniform throughout the whole length between the two points of support, must be efficiently anchored ;

(2) The steel bars must be placed at such a level that over the supports the height of their centre line above the underside of the cap is not less than $\frac{n}{3}$.

It will be realised that, with this " truss " distribution of forces in the cap, the upper part of the cap is in compression under the load and the lower part of the cap is in compression over the supports. In order to avoid overstressing the concrete in the latter position the centre of compression must be at a height of at least $\frac{n}{3}$ above the underside of the cap. Again, this centre of compression must be in a line with the centre line of tension in the bars, and hence this line too must be at least $\frac{n}{3}$ above the underside.

The cap shown in *Fig. 14* has been designed to meet these two conditions. As regards (1), the conditions of efficient anchorage in such a case as this are not very clearly laid down in the Recommendations for a Code of Practice, but it may be claimed that the single hairpin bend at one end and the pair of overlapping hairpin bends at the other, with which each pair of bars is furnished, constitute efficient anchorage. The bends have a radius of not less than six bar diameters, which, in view of the fact that the concrete cover is nowhere less than 2 in. and that there is consequently no danger of the concrete splitting, may be said to satisfy Clause 505 of the Recommendations.

Moreover the overlap given to the bends amounting to 45 bar diameters (in addition to the hooks at the extreme ends) may be said to satisfy Clause 604 (b) as it limits the bond stress to its permissible value of 100 lb. per square inch. Indeed it might be said that the conditions as to bond stress are more than satisfied, as it might be claimed that the overlapping bars, being curved, would develop a bond stress greater than the normal 100 lb. per square inch, and that the hooks at the ends of the bars are therefore not required.

As regards (2) it will be seen that the centre of the steel has been placed $6\frac{1}{2}$ in. above the underside of the cap, which is more than $\frac{n}{3}$ (which is only 4.1 in.).

This has been done to ensure 2 in. of concrete cover to all bars. The length and width of the cap have also been designed to ensure this cover, so that the final dimensions as designed are 7 ft. 6 in. by 2 ft. by 3 ft.

Some concrete has been saved by giving sloping " shoulders " to the cap over the supports. The resulting economy (if any) will be slight, but as the concrete in this portion is practically useless, if the cap is considered as a truss, it might as well be omitted.

It will be noticed that the vertical bars in the column and piles have been sloped to meet each other and to overlap within the cap, thus following the line of inclined compression in the concrete.

A cap on four piles can be designed in much the same way. For instance, *Fig. 17* shows a column 18 in. by 18 in. weighing with its load 160 tons, resting on

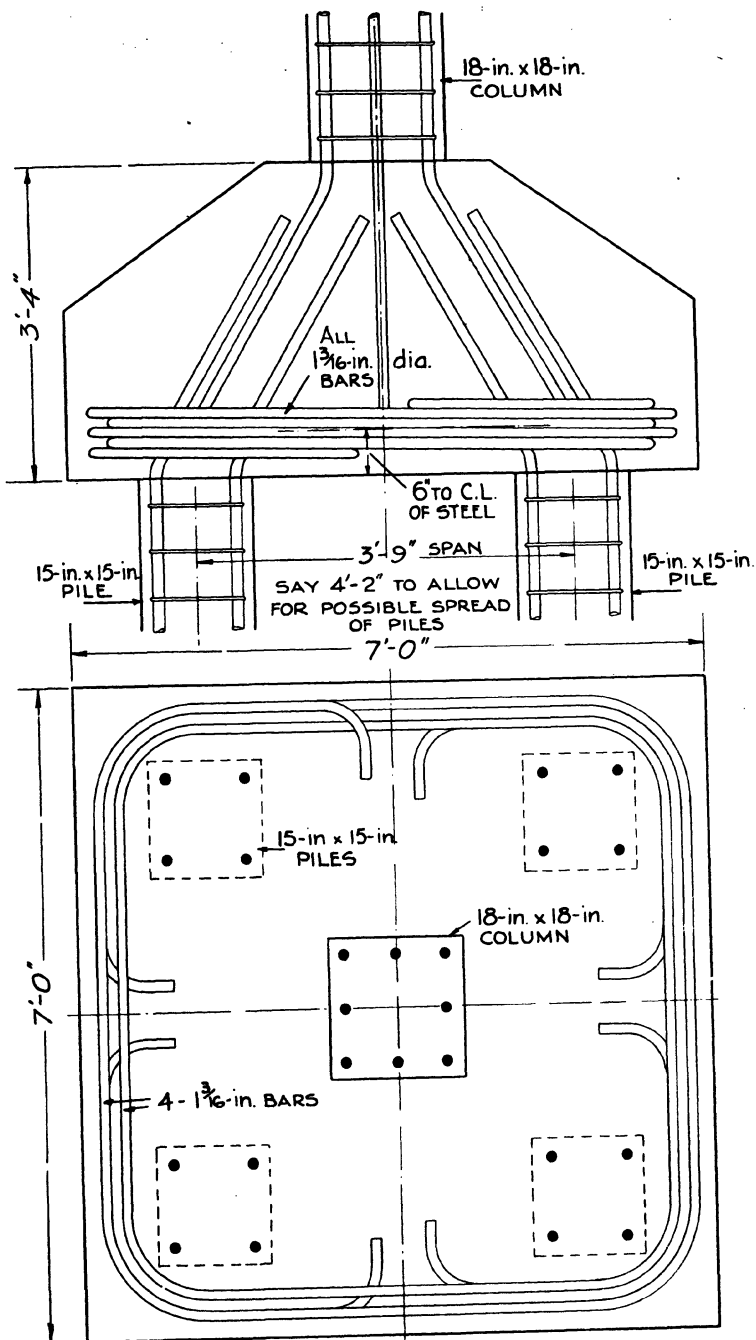


Fig. 17.—Four-pile Cap.

a cap which distributes the load on to four 15-in. by 15-in. piles driven 3 ft. 9 in. apart centre to centre. If we again assume that the concrete is to be stressed to 750 lb. per square inch and that the steel is to be stressed to 18,000 lb. per square inch, and if we further assume that half the column load is borne by each diagonally-opposite pair of piles, and that the diagonal span of 64 in. may be 70 in. owing to spread in driving, and that the width of the beam (for purposes of calculation) is the same as the diagonal width of the pile (say 21 in.), we find that the effective depth of the beam must be 32.2 in. and the area of steel 6 sq. in. It would, of course, be possible to provide steel of this area joining the two diagonally-opposite pairs of piles. But a more convenient and economical arrangement is to place the steel running along the four edges of the square slab and outside all the piles, as shown in *Fig. 17*. With this arrangement, although the length of steel required will be greater, its area need be only 0.707 of the steel if placed diagonally. This involves an area of 0.707×6.0 , or 4.3 sq. in. Four $1\frac{3}{8}$ -in. bars will suffice for this and, if placed as shown on the plan, it will only be necessary to have one overlap at each corner. This again is designed to have a radius of at least six bar diameters and an overlap length of 45 bar diameters plus a hook. The bars are placed in such a way that their centre line is more than $\frac{n}{3}$ above the base, and dimensions of the cap are finally chosen so that the concrete cover is nowhere less than 2 in. Thus designed, the dimensions of the cap are 7 ft. by 7 ft. by 3 ft. 4 in.

As in the case of the two-pile cap, concrete has been saved by sloping the shoulders of the cap, and the vertical bars of the column and of the piles have been bent over so as to overlap each other.

CHAPTER IV

SHEETING PILES, ANCHORAGES, AND RETAINING WALLS

As already mentioned, concrete piles are often driven in a row in order to retain an earth backing or for some such purpose. In such cases the piles are, of course, made as wide as possible so as to reduce the number to be handled and driven, the width being limited by the fact that increasing the cross-sectional area and weight of a pile increases the driving resistance and calls for heavier plant. If the piles are driven close alongside each other a tongue-and-groove arrangement as shown in *Fig. 18* is sometimes provided on the sides in order to keep the piles

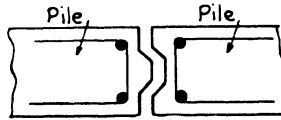


Fig. 18.—Tongue-and-Groove Pile.

in line. But it must be admitted that if the ground is such that the pile is liable to twist out of position this device will hardly restrain it. A more effective form of restraint is a wide flange steel joist placed between the piles. If these are to be eventually withdrawn a jack or extracting hammer may have to be used.

Piled Retaining Walls.

These can be built in several ways. The piles can be driven side by side continuously, or (for low walls) at intervals, the intervening spaces being filled with slabs (see *Fig. 19*). The slabs must be designed to transmit the lateral

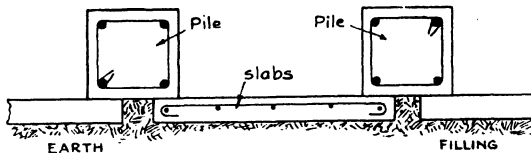


Fig. 19.—Piles and Slabs.

pressure of the earth backing behind them to the piles, which themselves must be designed to receive this pressure. The slabs must be taken down below lower ground level to a depth sufficient to prevent the retained earth passing underneath them.

If driven continuously the tops of the piles can form the top of the wall, in which case they are generally surmounted by a continuous capping, or the piles can be surmounted by a continuous wall of mass or reinforced concrete (see *Fig. 24*). It is sometimes economical to design the piles with a T or wedge-shape cross section (see *Fig. 20*) which increases their width without undue increase of weight or area.

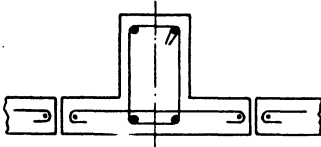


Fig. 20.—Tee-Shape Piles.

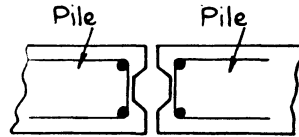


Fig. 21.—Sheeting Piles with Grouting Grooves.

Although, as mentioned before, it is customary to form the shoe of a sheeting pile with a rake so as to induce it to close up to its neighbour while being driven, it almost always happens that there is a slight opening between adjacent piles. This in many cases is an advantage as it prevents water accumulating in the earth backing behind the piling. But in cases where it is necessary that the piling shall be water-tight, or even perfectly earth-tight, this can be effected by forming a groove on both sides of every pile (see *Fig. 21*). After they have been driven the "tube" thus formed can be cleaned out and filled with cement grout or fine concrete. This can be done under water if the tube be filled with a "stocking" of sewn canvas before pouring the grout.

Anchorage of Walls.

Piled retaining walls are usually secured against forward movement by anchor ties spaced at intervals and placed at or near the tops of the piles. The ties are secured to a horizontal waling (see *Fig. 22*) which connects all the piles

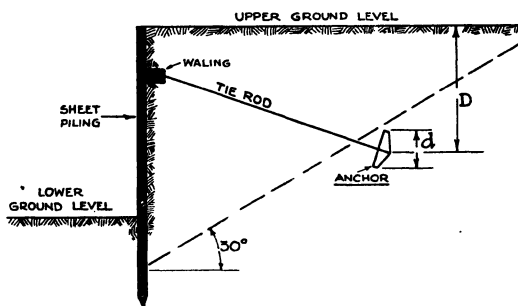


Fig. 22.—Anchorage of Wall by Tie Rod.

and transmits the thrust of the backing to the ties, and which therefore must be designed as a beam to resist the bending and shear stresses thus induced.

The lower ends of the piles may be secured by another set of anchor ties, but they are more usually restrained by driving them down into the ground a distance sufficient to develop the necessary earth resistance.

ANCHOR TIES.—The anchor tie is obviously in tension, and may simply

consist of one or more steel bars of sufficient section. To protect them from corrosion, especially if they have to be buried in damp porous earth, it is well to surround them with good concrete or to wrap them with canvas soaked in bitumen. Failing this they should be well coated with hot tar. The attachment of the ties at both ends is important. The bars should be finished off with hooks which

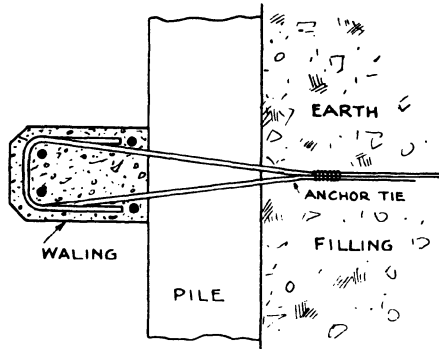


Fig. 23.—Connection of Anchor Tie to Waling.

surround the pile or waling or anchor to which the tension has to be transmitted (see Fig. 23) or else by some other efficient fastening such as a nut and washer. The hooks should be of sufficient length and width to comply with the Recommendations for a Code of Practice as regards hook diameter and overlap. As previously suggested, however, curved bars overlapping as in Fig. 23 should be able to sustain a higher bond stress than straight bars overlapping and should not need to be hooked at their ends.

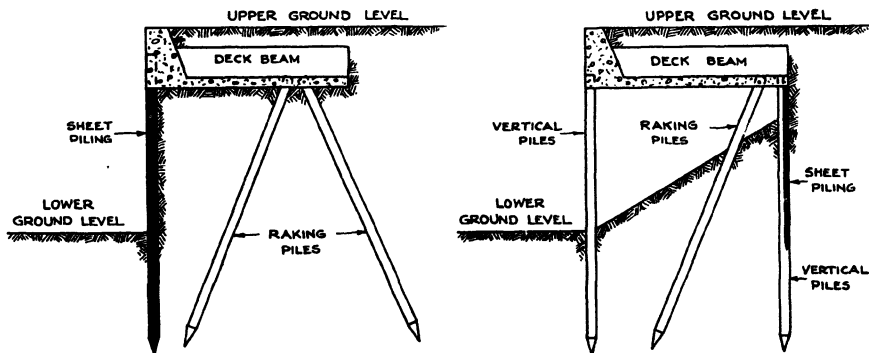


Fig. 24.—Piled Frame Anchorages.

ANCHORAGE BY PILED FRAMES.—Another way of anchoring a sheet-piled retaining wall is to secure it to a horizontal slab which is itself supported on vertical and raking piles. This type of wall is coming very much into favour. It can be designed in several ways which are mostly variants of the types shown in Fig. 24. This method of anchorage forms a piled frame and has the advantage that, unlike an anchor tie, the slab need not be carried back very far, as the

horizontal resistance to forward movement is supplied by the raking piles whose longitudinal resistance combined with the load on them produces a horizontal component. Moreover, as the slab itself receives the weight of the earth backing above, it relieves the piles below wholly or partly from the lateral pressure which would otherwise be imposed by that portion of the backing.

Stability of a Piled Retaining Wall.

In order to design piling for a retaining wall it is necessary to estimate (1) the lateral pressure of the earth retained by the piles, (2) the lateral resistance of the earth (at the foot of the piles) which resists forward movement, (3) the depth to which the piles must be driven to develop this latter resistance, (4) the pull on the anchorage near the top which also prevents forward movement, (5) the bending moment induced in the piles, and (6) the stresses in the piles caused by the bending moments.

Lateral Pressure and Resistance of Earth.

The estimation of the lateral pressure and resistance of earth, in spite of a very large amount of research work, is still a problem for which it is hardly possible to obtain an exact solution. For materials like clean gravel and sand, whose stability depends on their possessing a high degree of internal friction though lacking cohesion, Rankine's formula is frequently used, although it is not correct as it neglects the friction between the earth and the back of the wall. The effect of this neglect is to overstate the horizontal pressure, and the formula is therefore regarded as being "on the safe side." As will be shown later this friction must in fact be taken into consideration.

The formula which gives the horizontal intensity of pressure on a vertical wall is

$$p = wh \cdot \frac{1 - \sin \phi}{1 + \sin \phi}.$$

where p = intensity of horizontal pressure (tons per square foot)

w = weight per cubic unit of earth (tons per cubic foot)

h = depth below retained surface (ft.)

ϕ = angle of repose of earth, generally taken as 30 deg. for clean gravel and sand.

If 30 deg. be adopted for ϕ the formula becomes

$$p = wh \times \frac{1}{3}.$$

As regards the intensity of resistance to forward movement, Rankine's formula is also used for clean gravel and sand. It takes the form

$$r = wd^1 \cdot \frac{1 + \sin \phi}{1 - \sin \phi}$$

where r = intensity of horizontal resistance (tons per square foot)

d^1 = depth below lower ground surface (ft.)

ϕ = angle of repose of earth.

If $\phi = 30$ deg. the formula becomes

$$r = wd^1 \times 3.$$

If the material behind the wall is clay Rankine's formula requires considerable modification. Clay generally has a very low coefficient of friction, but on the other hand it possesses cohesion which gives it shearing strength which in a hard clay may be very considerable.

A modification of Rankine's formula devised by Mr. A. L. Bell, M.Sc., is often used for clay as it takes both friction and cohesion into consideration. It takes the form

$$p = wh \tan^2 \left(45^\circ - \frac{\alpha}{2} \right) - 2k \tan \left(45^\circ - \frac{\alpha}{2} \right)$$

where p = intensity of horizontal pressure (tons per square foot)

w = weight per cubic unit of earth (tons per cubic foot)

h = depth below retained surface (ft.)

α = angle of internal friction (deg.)

k = shear strength (tons per square foot).

The corresponding formula for resistance to forward movement is

$$r = wd^1 \tan^2 \left(45^\circ + \frac{\alpha}{2} \right) + 2k \tan \left(45^\circ + \frac{\alpha}{2} \right)$$

where r = intensity of horizontal resistance (tons per square foot).

d^1 = depth below lower ground surface (ft.), and other symbols as before.

Any units other than tons and feet may be used for these symbols provided they are used consistently throughout.

It must be remembered that these formulæ are for vertical walls. If the piling is battered the pressure will be less if the piling is battered backwards, and more if it overhangs its toe.

As regards the symbols, w , the weight of the earth, may be ascertained by experiment. Generally the weight of wet gravel or sand is assumed to be 120 lb. per cubic foot (0.054 ton per cubic foot) and of clay 125 lb. per cubic foot (0.056 ton per cubic foot).

As regards the symbols k and α , Mr. Bell in the paper in which he sets out his theory (Min. Proc. Inst. C.E., Vol. 199) describes an ingenious machine by means of which the values of these symbols can be ascertained for any given clay. He also gives values (*Table 5*) for certain typical clays which he examined.

TABLE 5.

	Shearing characteristics		Drop test	
	k (tons per square foot)	α (deg.)	12-in. drop (in.)	24-in. drop (in.)
(1) Very soft puddle clay . . .	0.2	0	1.3	1.5
(2) Soft puddle clay	0.3	3	1.1	1.3
(3) Moderately firm clay	0.5	5	1.0	1.2
(4) Stiff clay	0.7	7	0.9	1.0
(5) Very stiff boulder clay . . .	1.6	16	0.5	0.7

The drop test mentioned in the last two columns was made as follows. A steel ball $1\frac{1}{8}$ in. diameter and weighing 0.632 lb. was dropped on to a slab of the clay 3 in. in diameter and $1\frac{1}{8}$ in. thick from heights of 12 in. and 24 in., and the figures in the two columns record the diameters of the impressions produced by so doing. This test gives a rough and ready means of comparing the shearing characteristics of various clays.

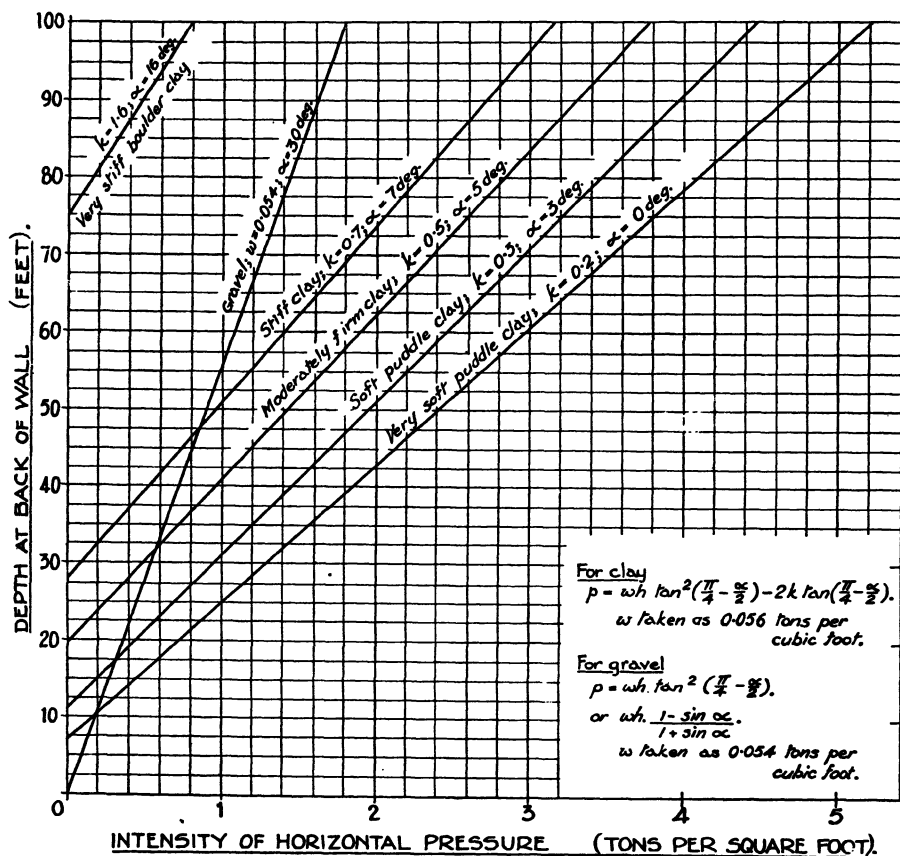


Fig. 25.—Relation between Depth and Horizontal Pressure.

It should be remembered that these formulæ do not profess to include any factor of safety, and also that clay, especially near a surface, is liable to soften by absorption of water. The choice of the value of symbols requires judgment, and indeed it must be admitted it is a matter of some uncertainty.

It will be found when using Bell's formula that it suggests that the lateral pressure of the clay for a few feet from the top of the wall is nil, or indeed a negative quantity. It is safer to treat this portion of the ground as if it were sand and to estimate the pressure accordingly.

To simplify calculations, Figs. 25 and 26 are curves giving the intensities

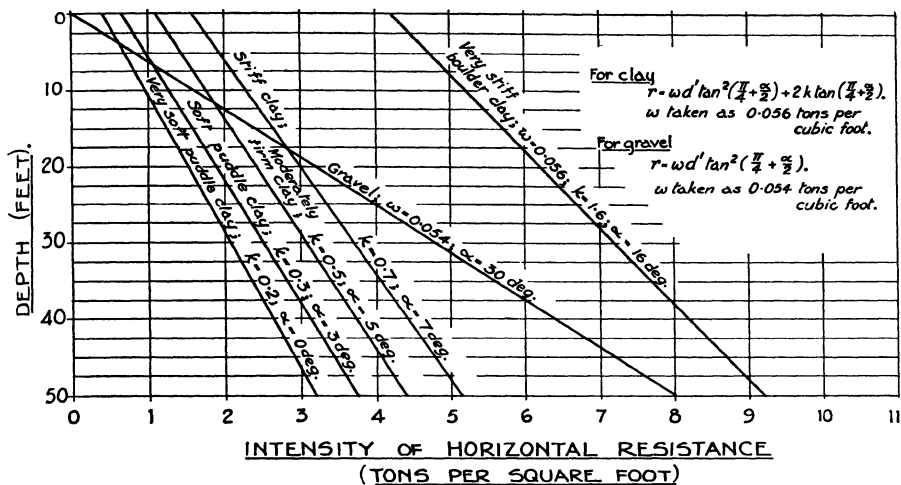


Fig. 26.—Relation between Depth and Horizontal Resistance.

of pressures and resistances at various depths calculated from the formulæ and using the values for the constants which have been suggested.

Length and Strength of Piles to Resist Earth Pressure.

Having decided the intensity of the earth pressure and the resistance at any given depth, it is then necessary to ascertain how far the piles must be driven in order that the earth resistance shall suffice to counterbalance the earth pressure and what pull is required on the upper anchorage for the same purpose. It is also necessary to ascertain the bending moments induced in the piles by these forces.

All this can most easily be done by a graphic method which can perhaps best be explained by an example such as is illustrated in Fig. 27. It represents a sheet pile retaining wall 20 ft. high backed by earth which is clean gravel for the first 10 ft., below that 14 ft. of soft puddle clay ($w = 0.056$ tons per cubic foot, $k = 0.3$ tons per square foot, and $\alpha = 3$ deg.), followed by firm clay ($k = 0.5$ tons per square foot, $\alpha = 5$ deg.). The piles are to be secured by driving them below ground and by anchor ties placed 5 ft. below the level of the retained ground. A surcharge of 2 cwt. per square foot is to be allowed for—or, say, the equivalent of 2 ft. depth of superimposed earth.

The first step in the solution of the problem is to construct a force polygon (Fig. 27) representing the horizontal pressure of the earth behind the wall and the resistance of the earth in front of it. The vertical line 0-14 represents the wall, and the upper and lower ground levels with the changes of strata are plotted to any convenient scale to intersect it. The intensities of lateral pressures and lateral resistance are plotted at these levels as horizontal ordinates. Lines are then drawn to join the ends of these ordinates, thus forming the required force polygon. The portion hatched in one direction represents pressure and the portion hatched in the other direction represents resistance. The portion cross hatched thus represents both and, as will be explained later, is ignored.

REINFORCED CONCRETE PILING

The intensities of the horizontal pressure or resistance at the various levels can be estimated by calculation, or in this case may be scaled from the diagrams given in the figure. The results are as follows :

PRESSURES.

Position	Depth below upper ground level (ft.)	Depth below surcharge level (ft.)	Intensity of pressure (tons per square foot) from Fig. 25
Upper ground level	0	2	0.04
Bottom of gravel	10	12	0.22
Top of soft clay	10	12	0.05
Bottom of soft clay	24	26	0.75
Top of firm clay	24	26	0.30
10 ft. below clay	34	36	0.75

RESISTANCES.

Position	Depth below lower ground level (ft.)	Intensity of resistance (tons per square foot) from Fig. 26
Lower ground level	0	0.65
Bottom of soft clay	4	0.90
Top of firm clay	4	1.35
10 ft. lower	14	2.00

It will be observed that immediately below the gravel the lateral pressure due to the clay is (according to Bell's formula) less than that due to gravel. As this is perhaps open to doubt the polygon has been "corrected" by continuing downwards the line of the pressures due to the gravel.

Now divide the force polygon into convenient strips by means of horizontal lines drawn through points 1, 2, 3, etc.; calculate the area of each strip 0-1, 1-2, etc.; this area represents the total pressure (or resistance) per foot run of wall for the depth of the strip. In calculating the areas below ground level the cross-hatched portion should be ignored, as this portion represents resistance neutralized by pressure.

The results in this case are as follows :

Portion	Pressures (tons)
0-1	0.16
1-2	0.27
2-3	0.38
3-4	0.49
4-5	0.61
5-6	0.82
6-7	0.80
7-8	1.21

Portion	Resistances (tons)
8-9	0.23
9-10	0.28
10-11	1.06
11-12	1.08
12-13	1.10
13-14	1.12

We can now form the vector polygon (*Fig. 27*) by plotting from right to left along any horizontal line the lengths 0-1, 1-2, etc., equal (to any convenient scale) to the pressures in the corresponding strips, and also from left to right the lengths 8-9, 9-10, etc., equal (to the same scale) to the resistances. Choose a pole P_0 at some convenient distance below the horizontal line and join P_01 , P_02 , P_03 , etc.

Now form the link polygon (*Fig. 27*) by projecting horizontal lines from the centre of gravity of each of the strips of the force polygon 0-1, 1-2, 2-3, etc. Through any point on the line through the strip 0-1 draw a line marked 0 parallel to P_00 and let it intersect a horizontal line drawn through the anchor tie at T . Also through the same point draw a line marked 1 parallel to P_01 , and similarly draw lines marked 2, 3, 4, etc., parallel to P_02 , P_03 , P_04 , etc. It will be noticed that at the lower portion of the curve thus formed the curve is reversed. From the point T as found previously draw a tangent to this reversed curve. This line, with the curve, forms the required link polygon. Its lower end is at such a level that the total lateral pressure of the earth above that level is equal to the total lateral resistance plus the pull of the anchor ties. In other words, it is the level to which the piles should be driven to develop the lateral resistance required. In this particular case the piles would need to penetrate (by scale) 5 ft. 6 in. below the lower ground level. The piles may in fact have to be driven somewhat farther than this for the following reason.

As previously mentioned, we have so far neglected friction between the earth and the back. This, however, exists and tends to drag down the piles, which must be driven to a sufficient depth to resist this tendency. It is not possible to give an exact method of estimating the vertical component of the pressure of various earths, but it may be said that it would probably not exceed one-third of the horizontal pressure of that part of the pile which is above lower ground level. The pile should therefore be driven until its resistance (as estimated by Mr. Hiley's formula) is greater than this vertical component.

In order to find the tension induced in the anchorage, draw through P_0 on the vector polygon a line P_0t parallel to the tangent through T on the link polygon. The length ot on the vector polygon will then represent the tension in the anchor tie to the same scale that the horizontal pressures and resistance are drawn on this polygon. In this case the tension on the anchorage scales 2.7 tons.

We have now to estimate the bending moment in the piles. This can be at once found from the link polygon, which is also a bending moment diagram. The horizontal distance between the tangent through T and the curve at any

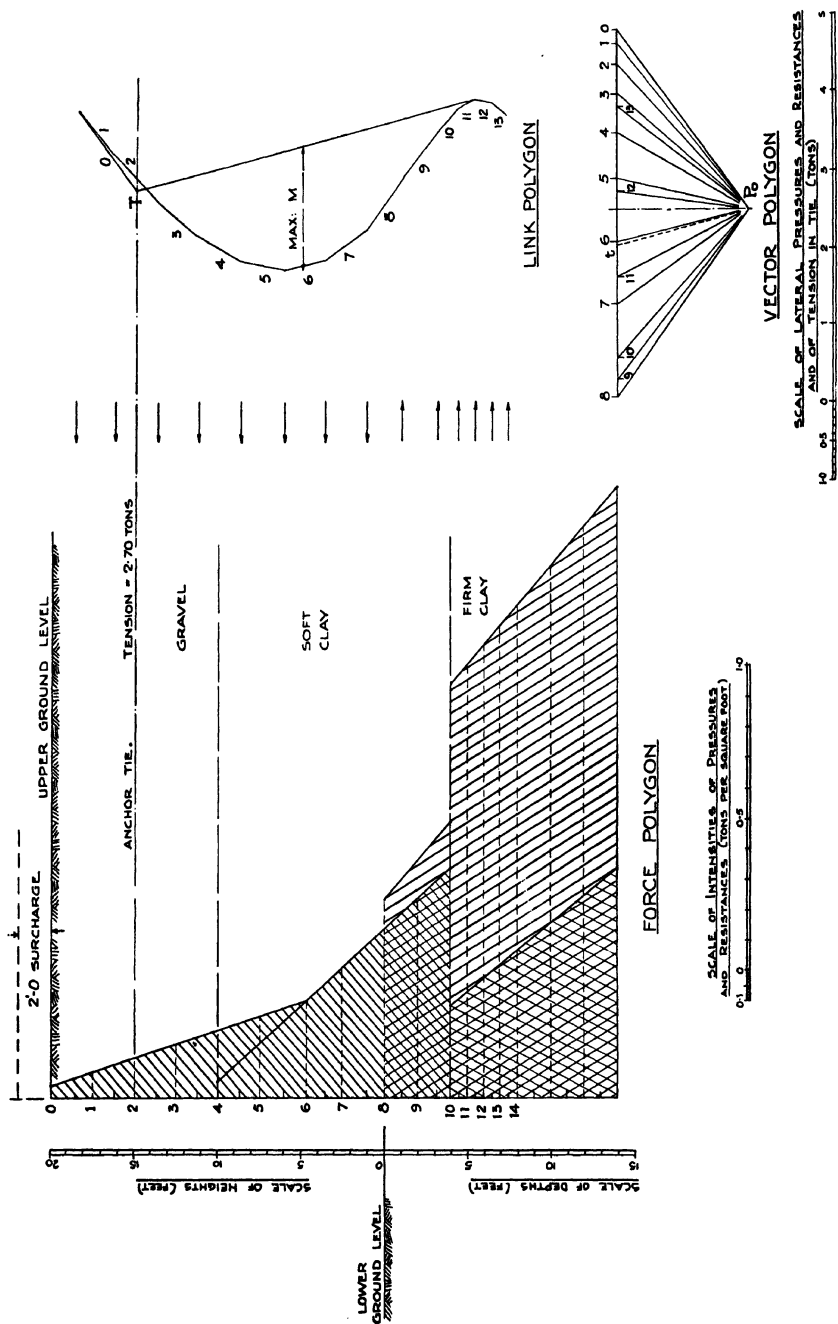


Fig. 27.—Graphical Design of Sheet Piled Wall.

level represents to scale the bending moment at that level, and where this distance is greatest the bending moment is at its maximum.

Its actual value is found as follows :

If the scale of heights on the force polygon be	A ft. to an inch
and the scale of loads on the vector polygon be	B tons to an inch
and the horizontal distance between the tangent and curve on the link polygon be	C inches
and the polar distance on the vector polygon be	D inches
the value of the bending moment is	

$$M = A \times B \times C \times D \text{ ft.-tons.}$$

In this case the maximum bending moment is found to be 11.7 ft.-tons.

It should be borne in mind, however, that the bending moment due to horizontal forces as thus found is increased to some extent by the fact that the friction between the earth and the back of the wall—which tends to drag the pile down—being an excentric load, produces a bending moment on the pile equal to the downward force multiplied by half the thickness of the pile, and this bending moment should be added to that produced by the horizontal forces.

In this case the bending moment due to vertical forces would be as follows :

Total lateral pressure between upper and lower ground level	4.74 tons
Vertical pressure, say, one-third of this	1.6 tons

Then the bending moment due to the vertical forces will be 1.6 tons $\times \frac{1}{2}$ thickness of pile (ft.), which should be added to the bending moment due to lateral pressure, etc., as previously found (11.7 ft.-tons).

When the tie is attached to the pile at a point below its head, the length of pile between the head and the point of attachment acts as a cantilever, and there is a negative bending moment between the head of the pile and some point below the tie. The negative moment in the pile attains its greatest value at the level of the tie, and its value is independent of any pressure or depth below it. For a triangular distribution of lateral pressure due to granular filling, and varying from zero at the pile head to p at the level of the tie, the maximum negative bending moment is

$$M = -\frac{px^2}{6}$$

in which x is the depth of the tie below ground level. This moment is usually small, unless the tie is attached at a point considerably below the upper ground level. The effect of lowering the tie is to increase the negative bending moment and to decrease the positive bending moment on the portion of the pile between the tie and the shoe.

The value of the negative moment may be calculated from the above formula or obtained from the graphical construction shown in *Fig. 27*. In the latter case the maximum value of the negative moment is represented by the horizontal intercept of the bending moment diagram by the line representing the tie, that is, by the short horizontal intercept between the lines 0 and 2 on the level of the tie. The point of contraflexure (that is, the point where the bending moment is nil) is indicated in this diagram by the intersection of line 2 with the tangent drawn to the reverse curve at 11-12.

Having thus estimated the total maximum bending moment, the stresses in the concrete and the steel can be calculated as explained under the heading "Stresses Caused by Lifting," and compared with the permissible stresses for the concrete and steel employed (see p. 21).

It may be said that the pile and slab type of design is suitable for walls up to about 15 ft. high and continuous sheet pile walls up to about 25 ft. high. The higher the wall the greater the bending moment on the piles and with h above, say, 30 ft., it becomes difficult to design a pile sufficiently stiff to transmit the lateral pressure to the lower ground and the top anchorage, unless additional anchorage on a lower level can also be provided.

The tables in Appendix III give certain information about walls of this type of varying heights and for various positions of the tie rod, assuming the earth backing to be a sand ($\phi = 30$ deg.). The information includes the total lateral pressure, the ground resistance, the pull on the tie rod, the minimum depth of pile below lower ground level, with the positive and negative bending moments in the piles, all per lineal foot of wall.

Length of Anchor Ties.

The length required to be given to an anchor tie, in order to ensure that the anchor buried in the earth backing shall be efficient, is a problem which is as yet full of uncertainty. It would seem necessary that the anchor should be placed behind any possible slope of rupture of the earth backing so that the wedge of earth between the piled face and the anchors shall be a stable one. It may be said that it is good practice to carry the anchor ties in firm dry soil to a point behind a plane through the foot of the wall and sloping at 30 deg. with the horizontal (see *Fig. 22*). If the ground is a soft clay the anchor should be carried back farther still. It will be noticed that this means that the lower an anchor is placed the shorter the tie may be. This seems reasonable, but the tie should not be too much out of the horizontal as its direction should be as nearly at right angles to the pile as possible. It should be remembered, too, that if it is not horizontal the tension in the tie will be increased and a vertical component tending to drag the pile down will be induced, and provision must be made for these forces.

Area of Anchors.

As regards the area and strength of the anchor, this matter, too, is full of uncertainty, but some engineers design an anchor by assuming that if buried in firm earth it will have a lateral resistance.

$$R = l \times d \times D \times 0.2$$

where

R = resistance (tons)

l = length of anchor (ft.)

d = depth of anchor measured vertically (ft.)

D = depth of centre of anchor below ground level (ft.)

The dimensions of the anchor are chosen so that R (as found above) will be not less than the pull in the anchor tie. The anchor itself is then designed as a slab loaded at one central point and uniformly supported.

Instead of anchor blocks, piles (generally driven with a forward rake) have

sometimes been used, but this is not an efficient arrangement as generally the resistance of the pile to forward movement cannot be used fully for fear of over-stressing it as a double cantilever.

Stability of Piled Frames.

As already mentioned, the resistance to lateral pressure in a piled frame of the type shown in *Fig. 24* is supplied by pairs or groups of raking piles supporting the back end of the slab. In the case of the simple wall here shown, we can calculate the vertical load imposed on the raking piles by the tail end of the slab and also the horizontal pull in the slab itself due to the lateral pressure of the wall backing. A force polygon will then give the tension and compression in the piles. The piles must be so spaced that these forces do not exceed a safe limit.

The permissible pull on a pile in tension is another matter of great uncertainty, but it is generally safe to estimate it at double its weight.

In estimating the lateral pressure on the face of this type of wall it is evident that the existence of the slab relieves the piles beneath it of pressure due to the earth above it unless the slab is very narrow compared with the height of the wall, and in any case gives some relief.

In this type of wall the slab, being loaded by the earth above it and supported by the piles, is subjected to bending moments which are calculated in the usual way and must be provided for.

A wall of this type (Hamworthy wharf) is described in some detail in Chapter X. A description is also given of dolphins at Southampton, which are designed on similar lines, to take the thrust or pull of a large floating dock which is secured to them.

CHAPTER V

JETTIES AND FENDERS

Piled Frames for Jetties, etc.

JETTIES and viaducts are now very commonly built with reinforced concrete piles surmounted by caps and beams and stiffened with horizontal and diagonal struts all of the same material, which is economical and very suitable for these structures. Reinforced concrete piles are especially favoured for jetties in rivers

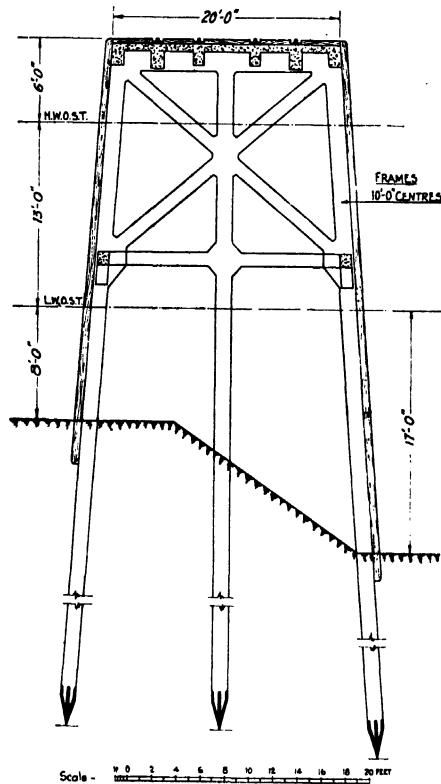


Fig. 28.—Early Jetty at Southampton.

and docks, where they can easily be driven from temporary staging or, under favourable conditions, from floating plant. The evolution of design of this type of jetty is well illustrated by *Figs. 28 and 29*, which are reproduced to the same scale. The first is a cross section of one of the very earliest jetties built at Southampton in 1901. The design is based on what was then a common

arrangement for timber jetties. It has the drawback that it is lacking in lateral strength, as there is no diagonal bracing below low tide level. The second is a cross section of a jetty built on the Thames in 1935 and is a far better arrangement from the point of view of lateral strength.

Such structures are liable to heavy stressing not only from the loads they carry, but also from the blows of heavy vessels when berthing alongside. The ideal arrangement to meet these conditions is not necessarily a structure which is so stiff that it will undergo no lateral movement, but rather one which will provide for a maximum of lateral movement without overstressing any member, in view of the fact that such lateral movement increases the range of distance over which the vessels' speed is reduced to nil, and consequently reduces the force needed to effect such reduction in speed. In a paper describing this jetty (Proc. Inst. C.E., Vol. 226) Messrs. C. P. Taylor and Oscar Faber point out that

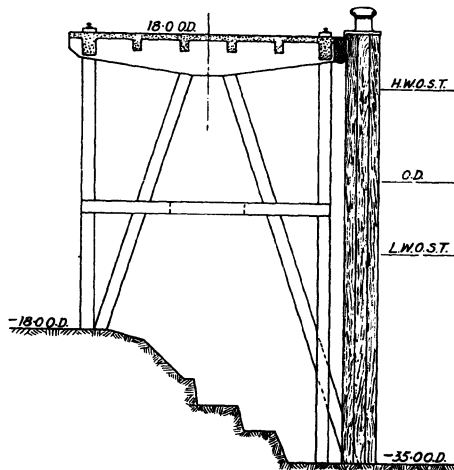
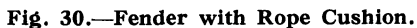


Fig. 29.—Jetty in the River Thames.

if a vessel of 12,000 tons displacement approaches the jetty with a speed of 1 ft. per second its kinetic energy will be 187 foot-tons. If this energy is absorbed after contact over a movement of 12 in., and assuming that the resistance of the jetty varies from nil at the start to a maximum at the end of the 12-in. movement, the maximum resistance to the blow will be 374 tons. They also point out that the horizontal resistance of each frame of the jetty, assuming that it is provided by the raking piles acting in tension and compression, may be estimated at 91 tons, and that the decking (except at the extreme ends) is strong enough as a horizontal girder to distribute a blow from a ship over five frames. In other words, the structure seems capable of resisting a blow effecting a horizontal point load of 455 tons, which is more than that of the vessel assumed. It must be borne in mind, however, that the 12-in. movement here assumed would be much more than could be made by the reinforced concrete work without serious damage, and that in order to permit of such movement some kind of flexible fender has to be introduced. The design of such fenders will now be considered.



Fenders for Wharves and Jetties.

It is usual to provide wharves and jetties with fenders which are invariably made of timber, whatever may be the material of which the main structure is composed. Not only when approaching a quay but even when lying alongside a vessel (which is rarely still) may damage that part of a structure with which it is in immediate contact. Timber is an excellent material to resist such damage, as it is tough without being too hard. It will bear a wonderful amount of crushing and abrasion before becoming useless, and when necessary can be easily and

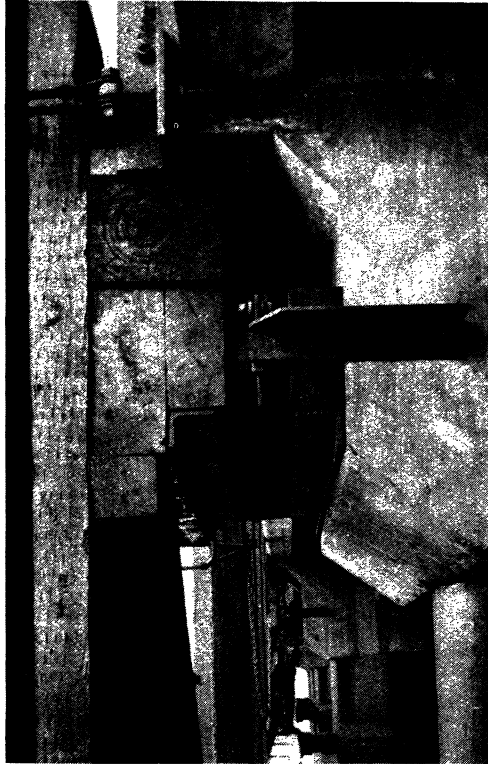


Fig. 32.—Rubber Cushion.

cheaply renewed. A very suitable timber is elm which, though soft, is very tough. It is, however, liable to attack from marine borers in sea-water, and where these abound a more durable timber such as greenheart is preferred.

The fenders should be secured to the structure in such a way that they can be easily taken off and renewed when necessary. If the fender is attached to the face of a pile which is not backed with earth, or otherwise stiffened, it should be arranged with a space of a few inches between the fender and the pile (except, of course, at selected strong points of support), so that a blow from a vessel even if it breaks the fender will not break the pile. A very good arrangement is that shown in *Fig. 29*, and to a larger scale in *Fig. 30*, where the fender is an inde-

pendent timber pile supported only by the ground at deck level. In this case a cushion consisting of a bag filled with rope is inserted between the fender and the deck. This, as previously explained, permits a considerable range of horizontal movement of the vessel as its speed is being reduced to nil by the resistance of the structure. Steel springs and rubber cushions have also been used for this purpose. *Fig. 31* shows a steel spring which is inserted behind the fenders at the Southern Railway Company's landing stage at Portsmouth. *Fig. 32* shows a rubber cushion used at the coaling jetty at the Fulham Power Station.

CHAPTER VI

PILING IN SEA-WATER

A GOOD deal of deterioration has been observed in piles and other reinforced concrete structures erected in sea-water. The prevention of such deterioration has been the object of much investigation and it may be fairly stated that, provided certain precautions are taken, reinforced concrete is likely to prove a material which is one of the most highly resistant to the action of sea-water. It is important to note that where such deterioration has occurred it has taken place not under water level but from about half tide up to several feet above high water. Apparently there is a tendency for the sea-water to rise in this portion of the structure by capillary attraction and to evaporate in the high and dry parts. If the concrete is pervious this action takes place readily. In fresh water little or no harm is done, but in sea-water the slow chemical action causes minute cracks in the concrete which accelerate the water travel and admit moisture and air to the steel. The latter oxidises, and the rust which is formed, being very bulky, bursts off the concrete cover even though the latter may have suffered no great loss of strength.

To prevent this action the most important requirement is that the concrete shall be impervious. This is best ensured by using a well-graded aggregate and a rich mixture such as $1 : 1\frac{1}{2} : 3$. Some engineers use an even higher proportion of cement, such as $1 : 1 : 2$, for the work above mean tide level. On the Continent deterioration is said to be greatly retarded in leaner mixtures such as $1 : 2 : 4$ by the admixture of such materials as trass or pozzolana.

Another important item in the production of impervious concrete is water curing. Experiments have shown that if concrete can be kept wet during the first few days after mixing the increase in its impermeability is most marked. With this object in view the side boards of pile moulds are sometimes made about an inch deeper than the pile itself, so that the concrete surface can be easily flooded with water while it is hardening. In any case deterioration is largely prevented by providing an ample thickness of concrete cover over the steel. If a rich mixture such as $1 : 1 : 2$ is used a cover of $1\frac{1}{2}$ in. may suffice, but with leaner mixtures it is safer to provide 2 in.

Some important evidence was gained during an experimental investigation into this subject by the Committee of the Institution of Civil Engineers on the Deterioration of Structures Exposed to Sea-Water, and the conclusions here quoted from their report of 1935 are worthy of careful study. They are given by permission of the Institution and of H.M. Stationery Office by whom the Report is published.

The experiments were made on small reinforced concrete piles, and also

on yet smaller plain concrete cylinders (intended for compression tests). Four types of cement were used, viz. ordinary Portland, rapid-hardening Portland, high alumina, and Portland blastfurnace. Three mixes, rich, medium, and lean, were adopted, trass being added to some of the lean mixtures and (in the case of the richer ones) two consistencies, dry and normal. Some pile bars were covered with 2 in. and some with 1 in. of concrete. The piles were partly immersed, some in sea-water and some in a brine three times as strong. Some of the cylinders were immersed in the brine and some in fresh water.

The following conclusions are based on experiments extending over a period of five years, and experiments are being continued for a further period of five years, when a supplementary report will be issued.

(1) Where a 2-in. cover of concrete was provided over the steel reinforcement, medium mixtures of proportions 1 : 1.67 : 3.33 afforded good protection against corrosion of the reinforcement.

(2) Of the cements used none showed any special advantage.

(3) The addition of trass in the case of lean mixtures proved advantageous. The reinforced specimens with this addition showed few or no signs of deterioration. Tests in which artificial pozzolanas were used did not provide definite evidence of the value of these materials.

(4) A dry mixture ($\frac{1}{2}$ -in. slump) proved slightly superior to a wetter mixture (2-in. slump), but the difference was not well defined. Practically no cracks were observed on the trowelled surface of the test-piles, owing probably to the increased impermeability of the concrete due to the trowelling.

(5) More piles showing cracks were found at Sheerness, where they were exposed in actual sea-water than at the Building Research Station in artificial sea-water of three times the normal strength. This was probably due to the more severe climatic conditions at Sheerness. The experiments, however, indicated that the behaviour of reinforced concrete in artificial sea-water was a very good guide to its probable behaviour in ordinary sea-water. Exposure on the Gold Coast has produced similar failures, but these are in most cases less severe than at the Building Research Station; this may in part be due to the fact that the specimens were older when exposed.

(6) Cracking has nearly always occurred after signs of rust have appeared on the surface of the concrete. It would seem, therefore, that the permeability of the concrete is largely responsible. The deterioration of the piles was chiefly due to the rusting and consequent expansion of the reinforcement, causing cracking of the concrete, rather than the attack on the concrete by sea-water. The deterioration of the piles as shown by cracking of the concrete occurred, with few exceptions, in the case of all mixtures above water level.

(7) Tests made with small cylinders have not given any guidance as to the occurrence of cracking in the reinforced concrete piles. Except with lean mixtures, the concrete of the test specimens was not materially affected.

(8) The experiments indicate that the concrete cover, so far as practicable, should be impermeable and that in the medium mixtures this can be obtained with a 2-in. cover.

When using reinforced concrete for structures such as groynes and jetties which are liable to suffer abrasion from moving sand, etc., it is still more important to use a strong and dense concrete with ample thickness of cover.

CHAPTER VII

SPECIFICATION FOR PILES AND MEASUREMENT

Specification.

THE specification for concrete piling is a matter requiring careful thought. It is essentially a description of the work and of the methods of carrying it out which is given by the engineer to the contractor. It lays down such items in the procedure as the engineer considers essential, but invariably leaves a good many to the contractor's judgment. For instance, a specification may lay down the quality of the materials used and the tests which they are to pass, the dimensions of the piles, the hammer blow and set required, and the age at which they shall be driven. But it may leave the contractor to select his plant and design his moulds and arrange his men. Provided that all work is carried out to the engineer's satisfaction this is a good arrangement. Of course, it presupposes that the contractor and his staff have had experience of this class of work, and indeed no contractor should be entrusted with such work unless he has not only experience but also good men and plant at his disposal.

The specification also lays down what risks are to be taken by the contractor and what risks taken (or what unforeseen expenses will be paid for) by the owner of the structure. For instance, it may require the contractor to drive piles of the dimensions given in the contract to a certain test set for the prices agreed whether the driving proves to be easy or hard. But it may also lay down that if he has to lengthen a pile in order to obtain the test set he shall receive extra payment. Again it may state that if the pile shall be broken or damaged at any stage the contractor shall replace it at his own cost. It should be borne in mind that while it is bad economy to impose too much risk on a contractor such risks as are imposed should as far as possible be clearly set out.

Typical clauses from a specification are here given. These will, of course, need to be varied to suit varying circumstances and to embody improvements in technique which are constantly being evolved by experience and research.

CEMENT.—The cement shall be obtained from approved British manufacturers and shall comply with the latest edition of the British Standard Specification for Portland Cement. Quick-setting cement shall not be used. The contractor shall supply free of charge a certificate from the manufacturer setting forth the tests that have been made on each consignment, with the date and place of manufacture. The bags shall be stored in a suitable watertight shed with a dry floor until the strength and other tests have proved it to be satisfactory for use. Each consignment must be kept separate and marked so that the older consignments can be used first.

When rapid-hardening Portland cement is to be used, it shall comply with the latest edition of the British Standard Specification for Portland Cement and in addition shall pass the following tests. The residue on a 180 by 180 sieve, equal to 32,400 meshes per square inch, shall not exceed 1.5 per cent. Cement and sand briquettes made as described in the British Standard Specification shall sustain a tensile stress of at least 300 lb. per square inch when 24 hours old and 550 lb. per square inch when 3 days old.

AGGREGATE.—The aggregate is to be perfectly clean gravel from an approved source. The gravel shall be separated into fine and coarse aggregate by washing it with fresh water through screens with openings $\frac{3}{16}$ in. and $\frac{3}{4}$ in. square, or $\frac{1}{8}$ in. and $\frac{1}{2}$ in. long mesh. Stones rejected by the latter screens may be crushed and re-screened.

REINFORCEMENT.—The steel for reinforcement shall comply with the latest edition of the British Standard Specification for Structural Steel and shall be of the quality known as "A" in that specification. All reinforcing rods shall be of the diameter specified. They shall be clean and free from oil, paint, mill scale, loose rust, etc. All bends shall be made cold to dimensions indicated on the drawings. All links shall fit closely round the main rods and shall be straight and taut between them. Intersections of main rods and, where directed, of main rods with links shall be securely bound with annealed soft pliable iron wire of No. 16 S.W.G.

MOULDS.—The moulds shall be sufficiently strong and rigid, and true to drawing, and so secured that they cannot move during concreting. The surfaces shall be smooth and the joints watertight to prevent escape of cement. They must be capable of being removed without jarring the concrete. Before use or re-use the surfaces shall be carefully cleaned and greased. When directed, fillets shall be fixed to give a chamfer on arrises. Moulds shall be carefully cleaned and freed from sawdust and dirt, and, if of timber, shall be wetted if necessary before concreting. Moulds shall not be removed until the concrete is sufficiently hard to admit of this being done without detriment. The engineer or his inspector shall decide when this shall be.

CONCRETE.—The materials shall be carefully gauged in proper boxes to give the correct proportions which shall be (by volume): Cement, 1 part; fine aggregate, $1\frac{1}{2}$ parts; coarse aggregate, 3 parts.

The aggregate shall be gauged by measuring it in proper boxes. The cement shall be measured by weight, 90 lb. being taken as the equivalent of a cubic foot. Great care shall be taken to ensure that the proportion of water shall be correct and under control, so that the concrete may be as uniform as possible in consistency and strength. To ensure this, a slump test shall be made daily, or oftener if necessary, at every mixing machine, and the quantity of water varied as required. This test shall be made by filling with the concrete a hollow metal cone, 12 in. high, 8 in. diameter at the base, and 4 in. diameter at the top. The concrete is to be filled in three 4-in. layers, and each layer punned thirty times with a $\frac{5}{8}$ -in. diameter pointed metal rod. On carefully removing the cone, the concrete should, after three minutes, fall or slump 3 in. or such other amount as the engineer or his inspector shall determine.

All materials shall be mixed in an approved batch machine driven by power. All concrete shall be placed in its final position within 30 minutes of the time

it was begun to be mixed. All concrete must be thoroughly rammed and packed around reinforcement and to ensure that it is homogeneous and free from interstices. The ramming shall not be confined to the upper surface of the layer being deposited, but the whole shall be thoroughly well rammed and consolidated. Where concreting has to be temporarily stopped, this shall be done as and where directed. When concreting has to be done over a face of set concrete, the face must be completely hacked off and cleaned as directed, and covered with neat cement mortar. Fresh concrete shall be covered with wet sacks for seven days, the sacks being frequently wetted, and also when necessary by straw hurdles to protect them from frost. No concrete shall be mixed when the temperature of the air or the aggregate is below 33 deg. F. Defective work shall be cut out and re-concreted as directed. When concreting has to be carried out after daylight the place must be adequately lit. The strength of the concrete shall be ascertained by testing 4-in. cubes. Cubes shall be moulded every day from each machine working.

MANUFACTURE OF PILES.—The piles shall be moulded to the dimensions and particulars shown on the drawings. The lengths of the piles may be varied by the engineer from time to time as the work progresses. The piles are to be made in suitable moulds resting on a firm and rigid bed, and must be truly straight. The piles must not be lifted until at least four days after moulding. In very cold weather, such time shall be increased as directed. The piles must not be driven until at least 21 days after moulding. All piles are to be carefully handled and marked with the date of moulding.

No cracked, spalled, crooked, or otherwise imperfect pile shall be driven.

DRIVING.—The piles shall be carefully and accurately pitched and driven vertically or to the inclinations and in the position shown on the drawings. They shall be driven to a final set of not more than 1 in. to 10 blows with a 3-ton semi-automatic hammer falling 3 ft., or such other equivalent sets as the engineer may direct, in the case of any other size and type of hammer.

LENGTHENING.—In the event of the pile being completely driven without the specified set having been reached, it shall be lengthened and re-driven. The head is to be stripped far enough to allow at least 3 ft. of the pile bars to project. The lengthening bars are to butt on to the exposed bars and both are to be evenly lapped by bars 6 ft. long and of the same diameter as the pile bars and securely bound to them by No. 16 S.W.G. black wire. New stirrups are to be fixed similar to those on the main body of the pile but spaced twice as closely. The Bill of Quantities includes provisional items for possible lengthening of piles.

The Specification issued by the Institution of Structural Engineers, although incomplete, is worthy of study. It contains special directions for cases where piles have to be driven in groups and where jetting has to be adopted, and other suggested clauses which are the outcome of recent research.

Measurement.

Quantities of piling work are estimated according to rules laid down by a Committee of the Institution of Civil Engineers appointed to report on the subject in 1933 (Inst.C.E., Report of the Committee on Engineering Quantities.

William Clowes & Sons, Ltd.). By permission of the Institution the following extracts from the report which apply specially to concrete piling are here given.

"The units of measurement for concrete piles are to be:

"Concrete	Cubic foot.
"Reinforcement	Cwt.
"Shoes, the weight being stated	Number of each size.
"Handling and pitching piles	Number.
"Driving piles to a given level	{ Number or linear feet of penetration.
"Driving piles to a given set	
"Cutting off or stripping heads of piles	Number.
"Stripping piles for junctions	Number.

"Concrete piles are to be measured to the lengths ordered by the engineer.

The concrete in the piles is to be measured by multiplying the cross-sectional area of the pile by the length of pile as cast, from the head to the butt of the shoe, no deduction being made for chamfers, tapered points, or the volume of reinforcement. The rates for concrete in piles are to cover moulds, separate items being provided for concrete in piles of differing cross-section."

The method of measurement of steelwork in concrete piles is as follows:

"Bar or rod reinforcement is to be measured according to the weight as calculated from the drawings or specified, the basis of the calculated weights being 40.8 lb. per square foot of metal 1 in. thick for rolled steel, no allowance being made for waste or rolling margin. The additional weight necessitated by specified overlaps is to be included in the measurement.

"Wire or other material required for binding or supporting the reinforcement is not to be measured, but its cost, as well as that of bending, hooking, and all other work in providing and fixing the reinforcement as shown on the drawings or specified, is to be covered by the rates for the bars or rods.

"The weights of bars or rods of differing sizes, but otherwise of the same description in regard to their position in the work, the labour involved, and the fixing, may be added together and given as one item, except that for rods or bars $\frac{1}{2}$ in. and under in diameter or size, separate items are to be given for each size.

"Separate items are to be provided for bars or rods 40 ft. in length and upwards, in steps of 5 ft.

"Separate items are to be provided for bar or rod reinforcements, steel links, helical bindings, and forks.

"When concrete piles are ordered to be lengthened in position after they have been driven, the materials used for lengthening are to be measured as previously described, an item being provided in addition for the extra cost of lengthening the pile, including stripping the exposed end and forming connection of new with old work.

"Handling and pitching piles are to be included as one item, separate items being provided for piles of the different lengths authorised by the engineer. For this purpose it is sufficient to group together piles of lengths up to 30 ft., exceeding 30 ft. but not exceeding 40 ft., and so on, the classification proceeding by steps of 10 ft.

"Separate items are to be provided for the driving of battered piles, double piles, sheet piles, piles in groups, and isolated piles, also for trial piles when these are required."

CHAPTER VIII

CONSTRUCTION METHODS

Arrangement of Pile-Making Yard.

FOR a big piling job space must be set apart for use as a pile yard, whose arrangement is governed to a large extent by the shape and area of the ground available and also by the rate at which it is desired that the piles should be driven. Obviously the casting of the piles must start first and must be carried on for a month, or whatever time is needed for the piles to mature, before driving commences. After that the pile casting must be carried on at least as fast as the driving, and, if faster, stacking space must be provided. If space is scarce it may be necessary to economise by stacking the piles one above the other. This involves lifting and transporting the piles, which again cannot be done until they are sufficiently matured. If, on the other hand, space is plentiful it may be preferred to turn the piles over sideways after they have been cast (so as to release the bases of the moulds) and leave them there until they are required for driving. With this arrangement the stacking ground is the same as the casting ground.

Whatever arrangement is adopted, it must include spaces for stacking materials, for building reinforcing skeletons, for casting, and for stacking piles, with roads to accommodate the wagons or lorries which bring the materials and the cranes which handle materials or take away the finished piles. The whole area should be planned so that, so far as possible, when any operation is begun it can be carried on without interruption until it is finished.

Some ways in which this problem has been solved are given in the descriptions of various works given later in Chapter X.

Aggregates.

A considerable area may be needed for storage of aggregate. The source from which the aggregate is to be obtained should be investigated at an early stage to make sure that the material is suitable and plentiful, and that it can be delivered to the storage area by available transport (rail, road, or vessel). Unless its suitability for concrete making is well known, it should be carefully examined to ascertain if it is reasonably free from clay and organic matter. The Recommendations for a Code of Practice give some suitable tests for this. If it is found to be dirty the aggregate must be washed so as to pass these tests. Cubes should also be made with it, preferably by an expert tester, to ascertain its concrete strength, using the mix likely to be used on the work. The Recom-



Fig. 33.—Assembling Reinforcement.

mendations give directions for making these cubes, and lay down that at 28 days High-grade cubes should give the following compression strengths:

1:1:2, 5,625 lb. per square inch.
 1:1½:3, 4,950 „ „ „ „
 1:2:4, 4,275 „ „ „ „

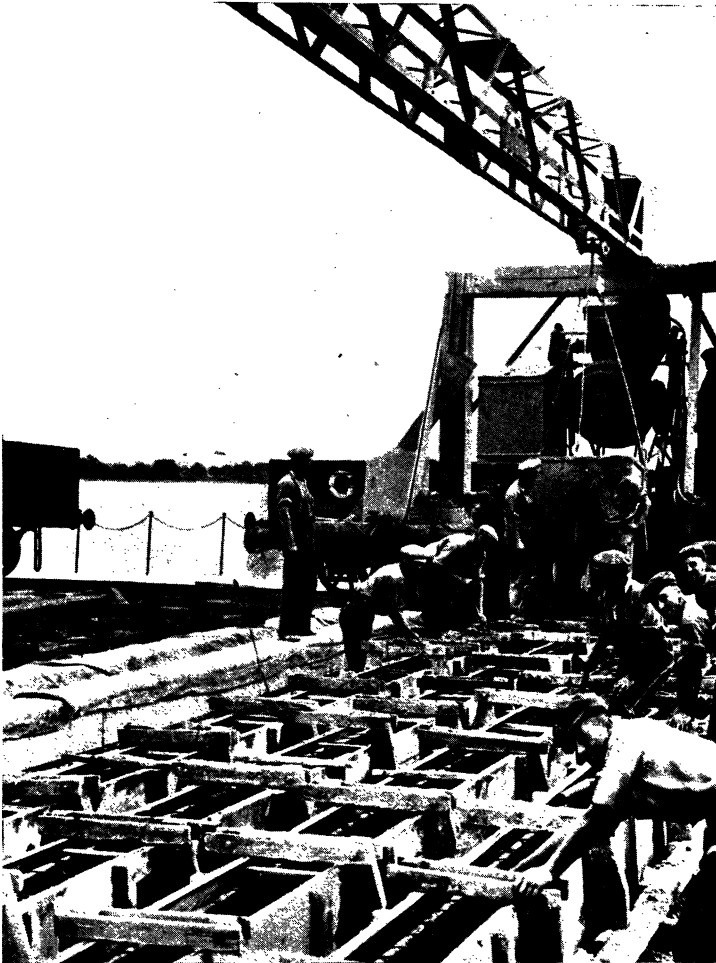


Fig. 34.—Timber Moulds for Square Piles.

These results are, of course, higher than are likely to be obtained from tests made on the works where the making of the cubes is not so carefully controlled. The strength of cubes which should be expected from works tests is given later.

Cement Storage.

On arrival at the site the cement should be stored under cover on a dry floor until the seven days' and other tests have been made. The bags forming

successive deliveries should be separated so as to make it easy to ensure that the cement is used in the order in which the consignments reach the site. Any cement found to contain hard lumps should be rejected.

Building Skeletons.

The reinforcement is built up into skeletons before being placed on the mould bases. For this purpose the main bars, which should be first cleaned from grease and loose rust, are generally placed on trestles spaced a few feet apart and connected by the links or spirals which are to bind them together and by the temporary or permanent spreaders which determine their relative positions (*Fig. 33*). The shoes should be wired on at the same time, taking care

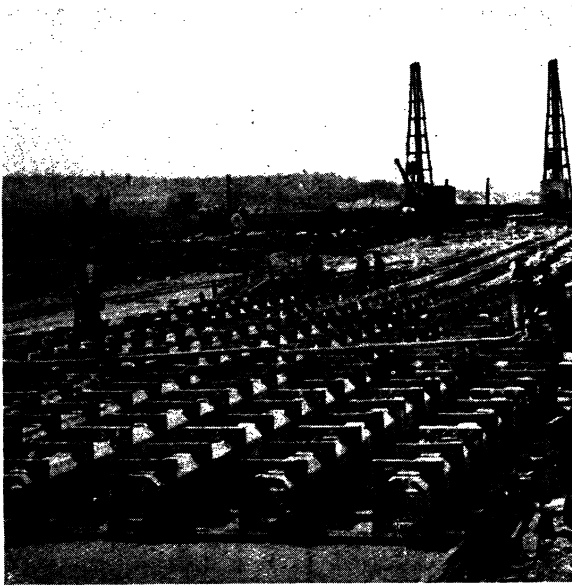


Fig. 35.—Timber Moulds for Octagonal Piles.

that the main rods rest firmly against them. All links and spreaders should be rigid and true before the skeleton is moved. If the piles are long the skeletons can be lifted by a crane using a spreader beam, and they should be secured in the mould so that they cannot move during concreting and so that the required cover is everywhere obtained.

Small steel ($\frac{1}{2}$ -in. diameter and less) should be kept under cover unless it is quickly used. There is no advantage in cement-washing the steel.

Moulds.

Timber is generally used for making pile moulds (*Figs. 34 and 35*) as they are then light to handle, but if they are to be used very often it may pay to line the moulds with thin sheet steel or even to make them of steel plate (*Fig. 36*).

The bases should be level and firmly bedded on well-packed sleepers, and if a large number of piles is required it may be worth while to lay a concrete floor (*Fig. 37*). The moulds should be true to drawing, strong, and rigid, and the surfaces which are to come in contact with the concrete should be wrought. All joints should be watertight to retain the liquid cement. In order to be easily removed the side shutters should be fixed by wedges and not nailed. If they are to be manhandled the side shutters should be made in short lengths. The fillets for forming the lower arris chamfers should be attached to the side shutters and the upper ones inserted after the last of the concrete has been poured. A

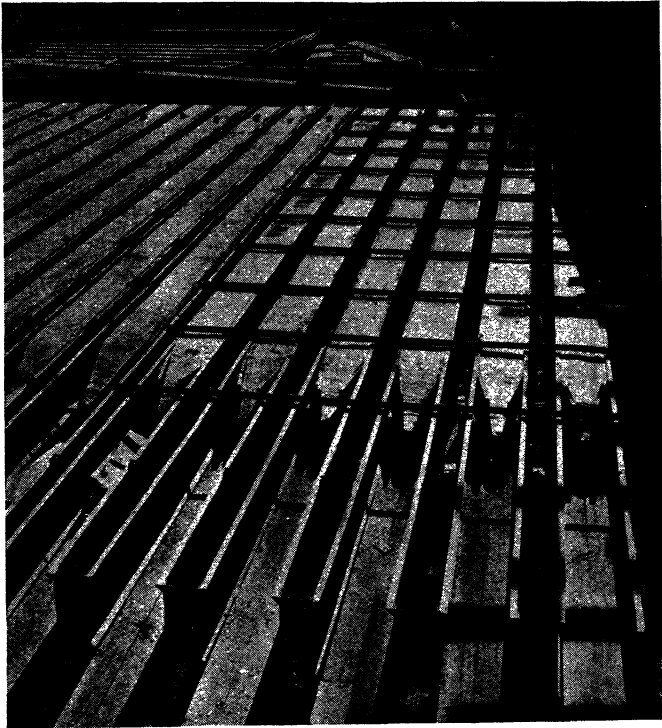


Fig. 36.—Steel Shuttering for Square Piles.

curing pond may be formed above each pile if the sides of the moulds are carried up above the level of the finished concrete.

The insertion of fillets to form chamfers is advisable as the concrete in sharp corners is often of bad quality. It is especially important to chamfer the arrises at the head of the pile as this helps to prevent damage while driving. Core tubes should be inserted at points where bolts will be needed for lifting or for securing the pile to the driving frame.

Before the steel skeleton is placed on the mould base the latter should be cleaned by a jet of compressed air or by brushing. The interior surfaces of the base and sides should be coated with grease or mould oil.

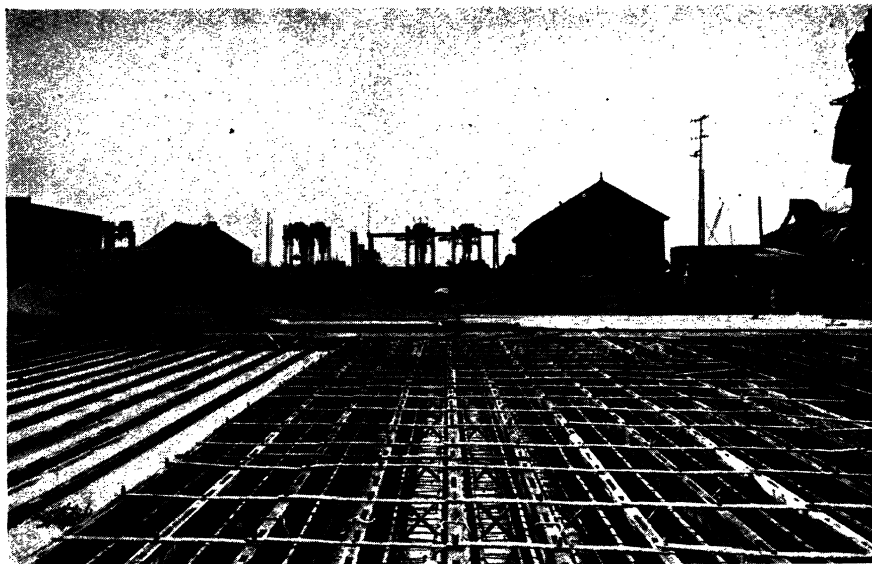


Fig. 37.—Timber Moulds Laid on Concrete Floor.



Fig. 38.—Travelling Gantry and Mixer.

Mixing Concrete.

Hand mixing for important work is now regarded as obsolete. If carefully performed it is a very tedious operation, and even then it is costly and unsatisfactory. For reinforced concrete work it is well worth while to use a power-driven mixer. The power may be steam, petrol, Diesel, electric, or any other



Fig. 39.—Depositing Concrete in Moulds.

available source. The best type of machine is a continuously rotating hollow drum mounted on rollers and provided with specially-designed baffle plates on its inner face so that the concrete ingredients are thoroughly mixed as the drum rotates. On each end of the drum is a large central hole; the dry materials are introduced at one end and the mixed concrete is discharged by means of a chute at the other end (see *Fig. 38*). Some small machines are made to discharge by tilting the drum, which is mounted on trunnions for the purpose.

Though the machine runs continuously, the materials are fed in a batch at a time. For this purpose on the feeding side a fixed hopper is provided which can be marked to indicate the required volumes of coarse and fine aggregate in a batch. The hopper may be replaced by a mechanically-operated side loader which lifts the charge of aggregate from ground level and pours it into the drum. Cement cannot be satisfactorily measured by volume and should be weighed. It is generally delivered by the makers in 1-cwt. bags, and these can be conveniently taken as $1\frac{1}{4}$ cb. ft. A definite number of bags should be used and the other materials measured to correspond according to the mix required. The water should be added after the dry materials have been fed into the drum; water is supplied from a tank which is automatically filled from a main supply, the exact quantity being capable of alteration if the resulting concrete proves to be too wet or too dry. After a sufficient number of turns have been given to the mixer the concrete is withdrawn on the discharge side of the drum. Barrows or skips placed under the chute receive the concrete, and are taken to the moulds.

It is important to secure a mixer of good make, and to take care of it. Lubrication should be attended to, and it should be well washed out every day after use with water and some clean coarse aggregate.

Mixers are made in various sizes to mix batches from 2 cb. ft. to 2 cb. yd. A $\frac{1}{2}$ -cb. yd. mixer is a favourite size for reinforced concrete work, but this, of course, depends on the output required. Mixers are usually designed to turn about 15 to 20 times per minute. The makers sometimes estimate that an average output for a machine is about 30 batches per hour, which includes half a minute for the actual mixing, but unless it has been ascertained that this time is sufficient for thorough mixing it is well to allow a whole minute. In practice it is not always possible to organise the feeding and discharging arrangements to deal with even 20 batches per hour. While, of course, the mixer must be adequate it is undesirable to use too large a machine, not only because it is extravagant but because the concrete suffers if it stays in the drum too long.

If the temperature is at or near freezing-point the work should be stopped unless the water, and the other materials if necessary, can be warmed so that the temperature of the concrete may be kept above 40 deg. F. until it has hardened.

Test cubes should be made twice a week from each mixer as suggested by the Recommendations for a Code of Practice, which gives the required strengths for High-grade concrete at 28 days as follows (see Appendix V) :

Mix	Compressive strength (lb. per square inch)
1 : 1 : 2	3750
1 : $1\frac{1}{2}$: 3	3300
1 : 2 : 4	2850

A slump test should be made at least once daily (see Appendix VI).

The following table is useful for estimating the quantity of materials needed to make 1 cb. yd. of concrete, with broken stone aggregate.

TABLE 6.

Mix	Cement (cwt.)	Fine aggregate (cb. ft.)	Coarse aggregate (cb. ft.)
1 : 1 : 2	8.8	11	22
1 : 1½ : 3	6.3	11.8	23.6
1 : 2 : 4	4.9	12.3	24.5

If ballast is used instead of broken stone the corresponding quantities are :

Mix	Cement (cwt.)	Fine aggregate (cb. ft.)	Coarse aggregate (cb. ft.)
1 : 1 : 2	8.4	10.5	21.0
1 : 1½ : 3	6.0	11.2	22.5
1 : 2 : 4	4.7	11.7	23.5

Conveying and Pouring Concrete.

On small jobs the concrete is conveyed from the mixer to the moulds by barrows or handcarts. Larger works are sometimes provided with a traverser (*Figs. 38 and 39*) spanning the casting platform on which a skip travels from end to end. If a skip is used it should be provided with a small spout so that the concrete will not spill outside the mould or else the space between the moulds should be boarded.

It is good practice to deposit the concrete in three layers in each mould, working from head to shoe of the pile, as this helps to expedite settlement of the concrete and prevent the occurrence of slight voids under the top steel.

All concrete should be placed in the mould within half an hour from the time of mixing, and should be well tamped with suitable tools throughout its whole depth. A good design of tamping tool has a narrow flat steel *L*-shape blade with which a slicing action may be produced. As an approximate rule there should be one man tamping for each cubic yard of concrete placed each hour. Vibration is now coming into favour as an effective method of consolidating concrete.

Curing and Stacking.

When Portland cement is used adequate curing is secured by keeping the piles wet during a period of at least a week after they have been cast. This may be done by covering them with damp sacking, watered at intervals, or better by flooding the forms, for which purpose it is necessary that the side shutters be constructed so as to extend above the level of the finished concrete. In hot climates flooding is the most satisfactory method of curing piles. Curing is essential whatever the weather conditions may be. Piles made with aluminous cement require to be cured most carefully, and it is essential to make the moulds thoroughly wet before placing the concrete and to keep the surfaces of the concrete thoroughly wet for twenty-four hours after it is placed. If they are kept insufficiently damp during the hardening period, these piles will prove unsatis-

factory. In cold weather the piles should be carefully covered with straw or sacking.

The number of the pile and the date of placing the concrete should be painted on every pile.

Age for Handling and Driving.

The age at which piles may be driven depends on the air temperature and other circumstances. It is customary when using good concrete made with ordinary Portland cement not to drive until they are at least a month old; it is sometimes necessary to extend this period to six weeks. With rapid-hardening Portland cement the period may be reduced to two or three weeks, and aluminous cement piles may be driven four days or even less after they have been cast.

In general, in reasonable weather and with a temperature of not less than 50 deg. F., the figures in *Table 7* may be used as a guide to the ages at which good quality piles may be freed from the moulds, lifted for stacking, and driven.

TABLE 7.

	Normal Portland cement	Rapid-hardening Portland cement	Aluminous cement
Stripping sides of moulds . . .	2 days	1 day	1 day
Lifting to stack	2 weeks	1 week	2 days
Driving	1 month	2 weeks	4 days

In cold weather these periods must be increased, and may have to be doubled.

Vibration of Concrete.

The method of consolidating wet concrete by vibrating it with electric or pneumatic tools instead of, or in addition to, tamping it by hand, is coming into favour. Although the technique of this process is not yet fully understood, it enables concrete to be consolidated in positions not readily accessible for hand tamping, and also enables a drier mixture to be used (resulting in a stronger concrete) with less risk of "air pockets" occurring in the concrete, which, if they do occur, more than neutralise the advantage of the dry mix.

The vibrating tools in use are of various kinds. For piles they usually consist of pneumatic hammers, or electric motors rotating eccentrics. The tools are rigidly clamped to the sides of the moulds at intervals, and cause them to "shiver" while the concrete is being poured.

The frequency of vibrations used may be high, about 6000 per minute, and the amplitude small (about 0.01 to 0.001 in.), or, by other processes of mechanical consolidation, a low frequency of about 150 shocks a minute and an amplitude of $\frac{1}{2}$ in. are used. Research work is being carried out as to how these and other matters should be varied so as to give the best results.

Pile Frames.

A pile frame is essentially a device to keep both hammer and pile in correct position and direction and to support the lifting sheaves. It is made of timber or steel and consists essentially of a pair of "leaders" (usually upright) to which

the hammer and the pile are secured in such a way that they can only move up or down. The leaders are usually braced at intervals horizontally and diagonally to four long raking struts (*Figs. 40 and 41*) which stiffen them in two directions and prevent them overturning. The frame is provided with decks at intervals of a few feet on which a man can stand to secure or release the pile and hammer.

The frame must be stable and robust to stand the racking stresses induced by the hammering and by the pile itself; the latter often tends to be distorted



Fig. 40.—Pile Frame.

out of its true line and must be restrained by the frame. To increase its stability the winch which lifts the pile and hammer is mounted on the base of the frame.

If it stands on the ground the height of the frame must obviously be the length of the pile plus that of the hammer plus clearance. If it stands above ground (on a barge or on a stage) its height may be less, but if the pile is to be driven below the base of the frame the leaders must project downwards so that the hammer and pile head can always be secured to them.

Sometimes frames are fitted with telescopic leaders sliding on the fixed leaders, thus allowing the latter to be lengthened as required. This arrangement is

particularly convenient when driving piles in a timbered trench, as the piles can be driven until they are well below the top of the trench and the telescopic leaders can then be drawn up to permit moving the frame without fouling the trench struts.

Another arrangement which has many advantages is to form the pair of



Fig. 41.—Pile Frame.

leaders into a stiff mast (*Fig. 42*) which can be detached from the frame, but when in use can be secured to it at any level, and either in a vertical position or inclined in any required direction.

The leaders are fitted with a cap for mounting the head gear, which comprises sheaves for lifting the pile and hammer, and shackles for attaching guy ropes to steady the frame. A good arrangement is to have three sheaves, one mounted

centrally over the leaders for the hammer and one on each side of the leaders, so that the pile may be lifted and pitched from whichever side is more convenient.

The bottom deck or main platform is usually mounted on travelling gear which most frequently takes the form of four double-flanged wheels so that the frame can travel on rails, the wheels being arranged to swivel and each being



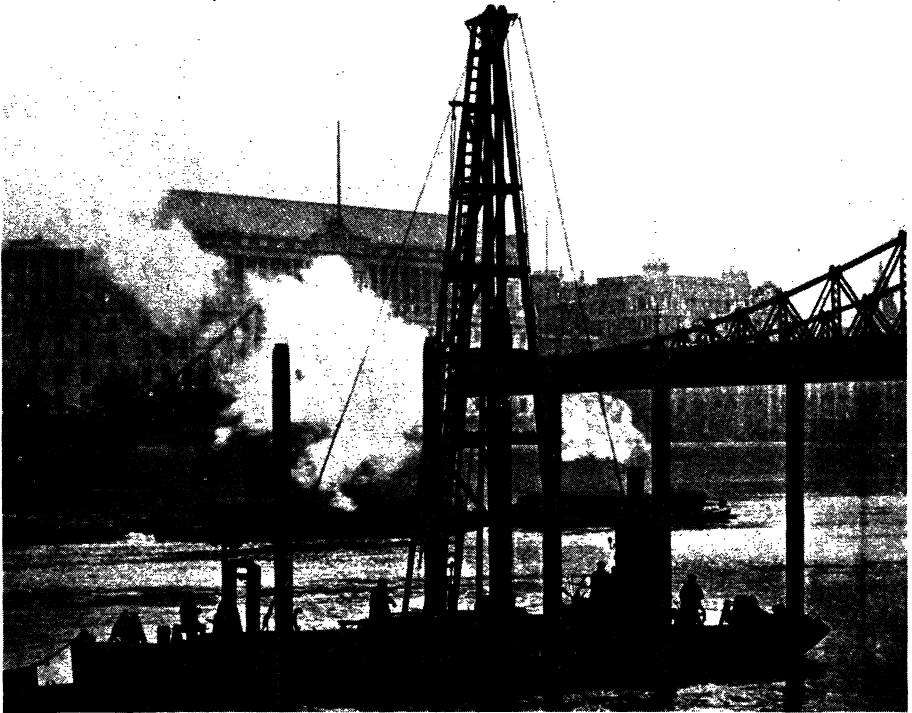
Fig. 42.—Pile Frame with Mast Leaders.

provided with a fixed jacking screw which serves to lift the wheel when required and to adjust the position and level of the frame.

Instead of wheels, the frame may be mounted on semi-circular saddles resting on two long steel rollers travelling on sleepers. This arrangement admits of easy movement not only forward but also sideways within limits. If very flexible movement is required the frame is sometimes mounted on a turntable.

For driving piles in long rows (as for a jetty or viaduct) the frame is often mounted on the outer end of a cantilever platform whose inner end rests on rails and bearers laid on the piles already driven, thus obviating the necessity for temporary staging.

For driving piles in water the frame is often mounted on the deck of a barge (*Fig. 43*) which also contains all other necessary appliances such as winch and boiler for lifting and driving, with chains and anchors for fixing the position of the barge. Pile driving from a barge is convenient in many ways. A drawback



• **Fig. 43.—Driving Piles from a Barge.**

to its use is that even in quiet water it is not possible to locate the piles with great accuracy, owing to small movements of the barge while driving. For this reason the piles of structures such as jetties and quays are often driven by a frame mounted on temporary staging made of timber piles which have themselves been driven by a frame mounted on a barge.

Winches.

Whatever type of hammer is used the pile frame is usually equipped with a winch and, if the latter is worked by steam, with a vertical boiler. The winch is generally fitted with two barrels for lifting the hammer and the pile respectively. If a drop hammer is used it has to be operated at high speed, whereas the pile

needs to be lifted slowly. This may be effected by lifting the hammer on a single-line rope and the pile with a tackle. Alternatively the barrels may be geared for lifting at different speeds. To facilitate working with a drop hammer the barrels should be freely mounted on their shafts and driven through friction clutches which can be engaged or released quickly without stopping the engine. The barrels should operate independently; they should be fitted with brakes and with a ratchet and pawl for holding the load, and with a warping drum on the barrel shaft extension for moving the pile frame, etc. (*Fig. 44*).

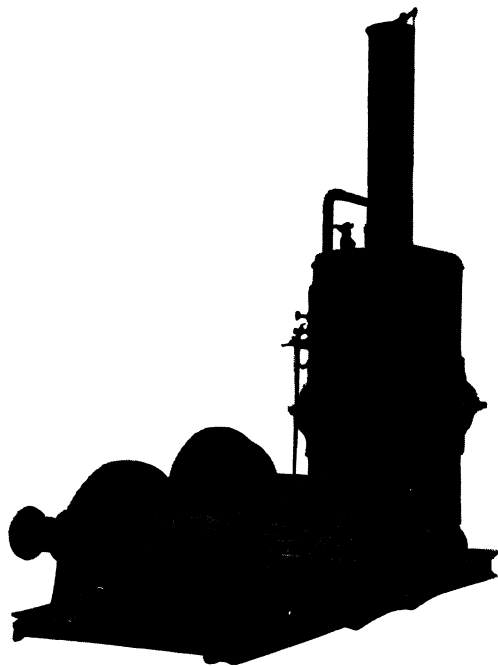


Fig. 44.—7½-in. by 10-in. Friction Winch.

Drop Hammers.

Drop hammers are blocks of cast-iron (*Fig. 45*) prismatic in shape and weighing usually from 1 to 6 tons. They are fitted with lifting eyes and provided with cuds and T-headed bolts for securing them to the frame leaders. The drop hammer is worked by the winch, the height of drop and the rate of blows being under the control of the driver. Although not so fast as a steam hammer it is regarded as a very useful and efficient tool. It requires considerable skill to work it quickly.

Single-acting Steam Hammers.

Single-acting steam hammers (*Fig. 46*) are coming into favour because, although they use more steam than a winch-driven drop hammer, they are faster in action, simpler to work, and impose far less stress on the pile frame and ropes.

The hammer is an inverted steam cylinder, the piston and piston rod resting on the pile head throughout the operation. Steam is used to raise the cylinder, which drops by gravity directly the steam is exhausted. The stroke can be varied up to its limit (usually 4 ft. 6 in.) and the weight ranges from 1 to 6 tons. This type of hammer can be worked by compressed air. The pressure is usually 80 lb. per square inch (say 100 lb. per square inch in the boiler.) About thirty blows per minute can be given with this type of hammer.



Fig. 45.—Driving with a Drop Hammer.

The admission of the steam is controlled by a lever worked by a man below. When steam is cut off the exhaust is automatically opened and the hammer drops. Hence this type of hammer is often spoken of as semi-automatic, but the term is misleading. Care must be taken with this type of hammer not to admit steam before the hammer has fallen, or the effect of the blow will be reduced.

Double-acting Steam Hammers.

In the double-acting steam hammer (*Fig. 47*), as its name suggests, a heavy piston (which is the hammer) is both lifted and forced down by steam pressure. The cylinder which rests on the pile head is also designed to be very heavy (so as not to be lifted by the steam pressure on the down stroke). The valves are controlled by the piston movement and thus the machine is entirely automatic

and the blows are delivered in very rapid succession—about 100 to 300 per minute according to the weight of the hammer.

One advantage of the double-acting hammer is that it can be used without a pile frame. If piles can be held in position by horizontal bracing or otherwise the hammer can be kept at work on it by suspending it from a crane.

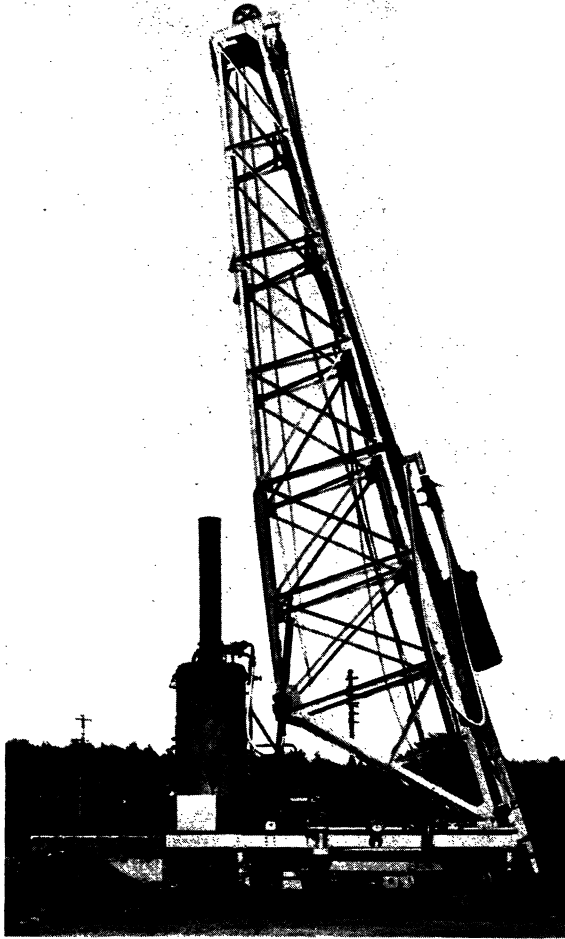
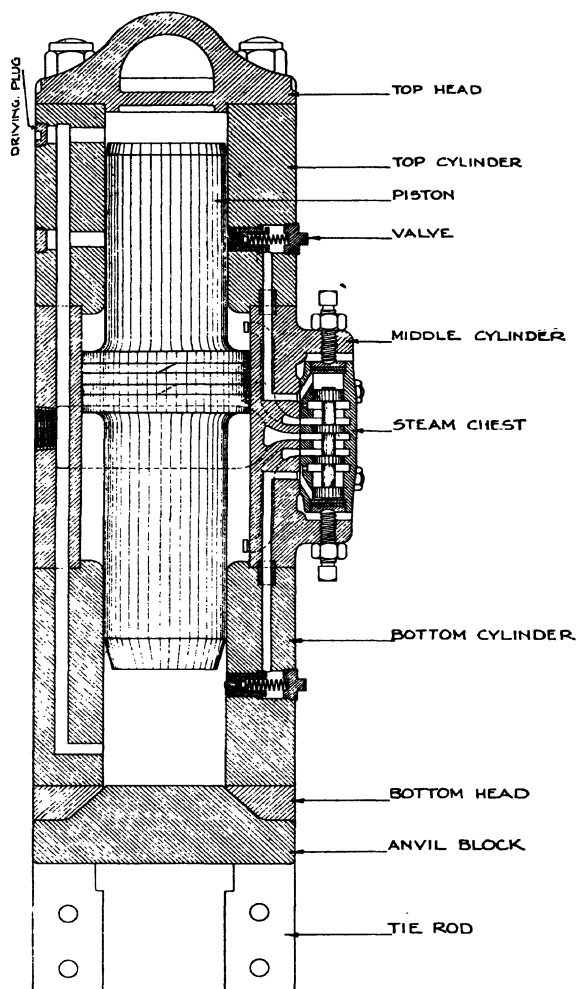
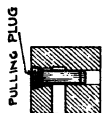


Fig. 46.—Single-acting Hammer.

Another advantage is that under the right conditions the double-acting hammer can be used for driving piles under water; yet another advantage is that it can be adapted for use as an extracting hammer.

Generally it may be said that for light concrete piles, and even heavy ones where hard driving is not expected, the double-acting hammer is to be preferred as it is automatic, simple to handle, and with its rapid blows keeps the pile "alive." But owing to its small stroke it is not so suitable for driving against a high re-



SECTION ABC

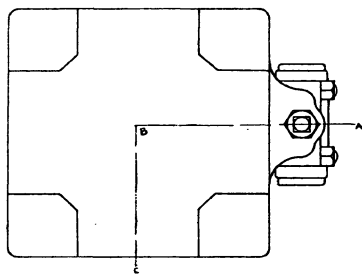


Fig. 47.—Double-acting Hammer.

sistance, and the single-acting steam hammer or the winch-driven dumb hammer is in this case the more effective. The heaviest double-acting hammer stocked by the British Steel Piling Co., Ltd., has a total weight of 6 tons, a stroke of 20 in., and delivers a blow of about 8 ft.-tons; but a single-acting steam or drop hammer of 3 tons weight and 4-ft. 6-in. drop delivers a blow of about 13 ft.-tons and is therefore more effective with hard driving.

Helmets, Cushions, and Dollies.

If the driving is not too hard and the head of a concrete pile is strengthened with ample transverse reinforcement, the hammer may be allowed to fall directly on to the head of the pile. But for reasons given in Chapter II it is usually necessary to interpose a "cushion" or soft packing between the hammer and the head of the pile to obviate excessive stresses from the hammer blow. This is usually effected by covering the head of the pile with a helmet which consists of a short steel casing, the shape of and large enough to cover the head of the pile, and divided into halves by a steel diaphragm. *Figs. 48 and 49* show two such helmets, one of cast steel and the other built up of mild steel plates. The cushion is generally placed on the head of the pile beneath the diaphragm, and above it is placed a short timber "dolly." The cushion usually consists of some yielding material such as felt, coir mats, coiled hemp, rope, sacking, asbestos fibre, or sawdust in bags. As mentioned on page 19 it is important that the cushion should truly fit the top of the pile in order that the stresses from the hammer blow may be evenly distributed, and for this reason sawdust, which quickly takes up the shape of the pile head, is often used. Even sawdust, however, after repeated hammering becomes so hard as to be ineffective as a cushion, and a packing which will not change in this respect is greatly needed.

The dolly in the upper part of the helmet is usually a piece of whole timber a foot or two in length, or timber boards placed one above another with the grain in alternate directions. If the head of the pile has to be driven lower than the hammer can fall the dolly must be a long one, but where possible this should be avoided as the wear and tear on dollies and the expense of renewing them are often serious matters. For long dollies a timber like elm, which is tough and not too hard, is often used, but if the driving is hard a stronger timber such as greenheart is preferred.

Jetting.

Sinking piles by means of water jets is a fairly common practice and is very effective in suitable soils. The most suitable is clean sand, through which piles can be quickly lowered without the use of a hammer. In other materials, such as loamy sand and soft clay, the jet forms an excellent aid to the hammer, but in materials like coarse gravel and very hard clay the jet is practically useless.

In the majority of cases where jets have been used they have been carried down to the foot of the pile by an independent tube, or preferably by a pair of tubes. This arrangement has the advantage that the jet can be taken down, if desired, below the point of the shoe and that the number of jets can be easily increased from one to two or even three if necessary. In some cases the jet has been taken down by means of tubes attached to the piles or, in the case of concrete piles, by tubes cast inside them (see *Fig. 1*, page 4).

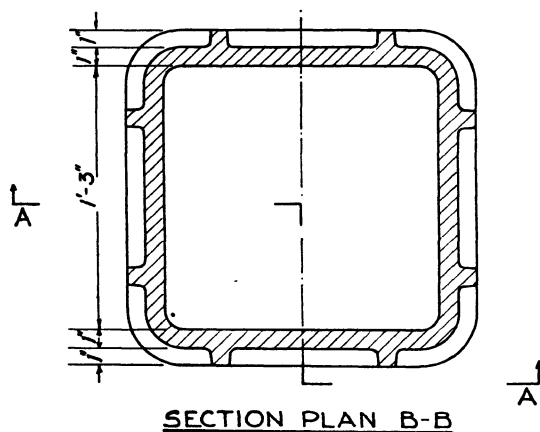
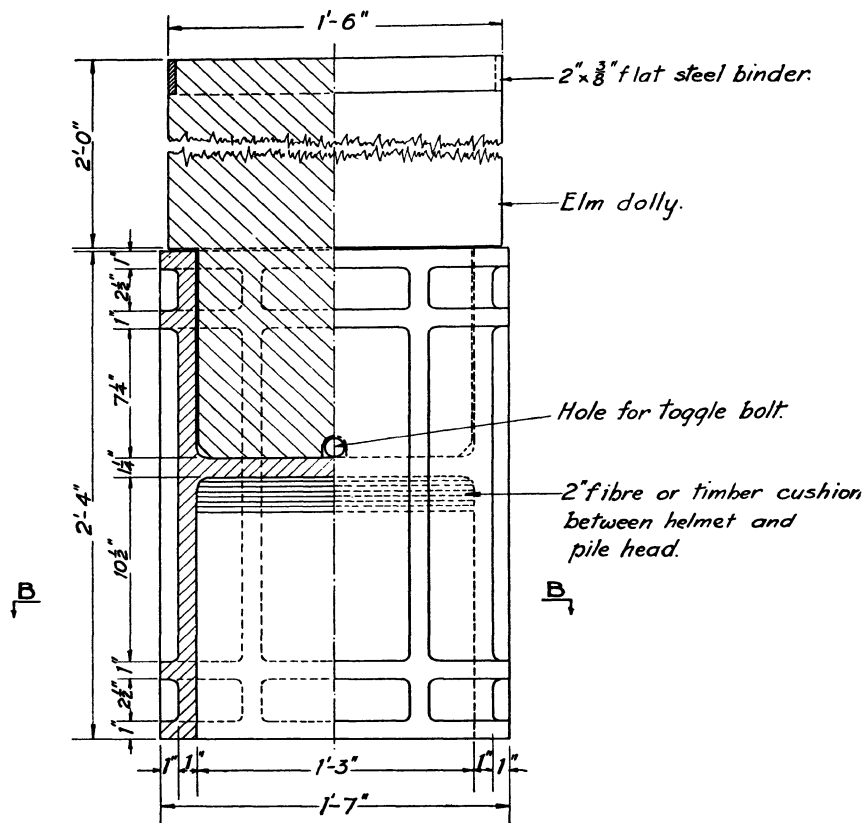


Fig. 48.—Detail of Cast Steel Pile Helmet.

A favourite size of tube is about $1\frac{1}{2}$ in. to 2 in. with a $\frac{3}{4}$ -in. to 1-in. nozzle at the outlet. To feed this the water-jet pump should be of the duplex ram pattern—capable of delivering, say, 3000 gallons of water per hour at a pressure of 120 lb. per square inch. If more than one tube is used the pumping plant must be increased accordingly. If the jet is effective and the driving is thereby lightened the boiler which provides steam for the hammer may suffice to work the jetting pump.

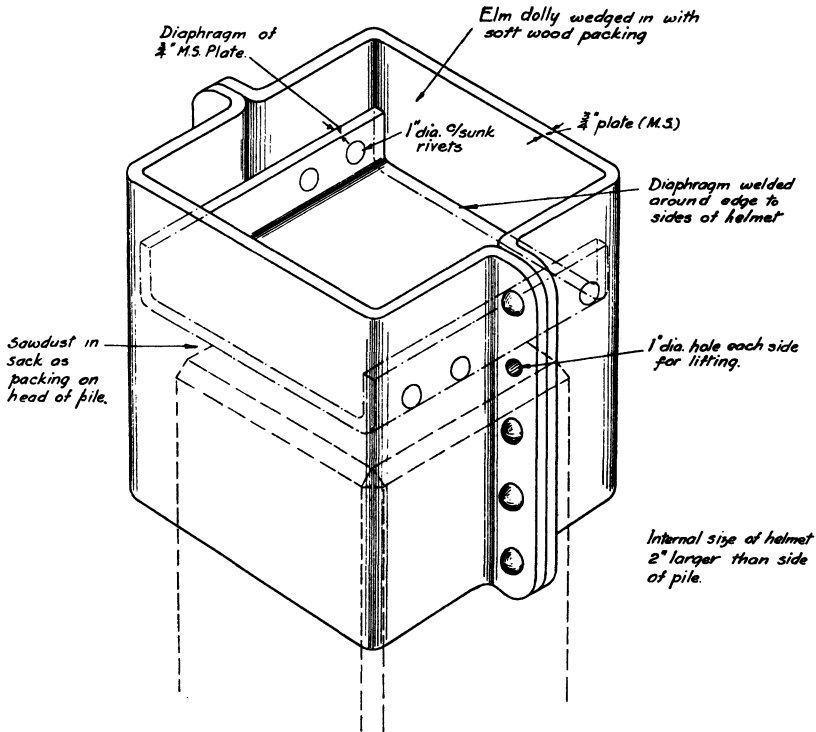


Fig. 49.—Mild Steel Pile Helmet.

If an independent jetting tube is used it should be withdrawn before the water is turned off, otherwise the sand may close around the pipe and hold it fast. If the pump is at some distance from the pile the connecting main should be of ample size, say, equal in diameter to the pump discharge. If the jet is being used as an auxiliary to the hammer it should be shut off before delivering the final blows for the test.

Lengthening and Re-driving.

If a pile has been driven down to ground level and its set for a given hammer and drop still exceeds the required standard it can be lengthened and re-driven. For this purpose it is usual to cut away a sufficient length of the concrete, leaving the bars exposed. New bars are then erected in continuation of them, the butt joints being covered by lengths of bar of the same diameter securely wired to old and new. Shuttering is then erected around the bars and the whole con-

creted. The length of pile to be cut away is usually about 3 ft., and the splice bar should be twice this length. The butts should be true and in line. The links should be similar to those in the original pile, but closer. At the head of the pile the stirrups should, of course, be closed in as before.

In order to reduce the delay while the new pile head is maturing it is frequently moulded in high-alumina cement concrete, in which case the pile can be re-driven in a few days.

Pile Connections.

It almost invariably happens that the head of the pile has to be cut away in order that the bars may project into the cap or beam above it to a sufficient extent to develop safe bond stress equal to the estimated compression in the bar, whatever that may be. The concrete can be cut away with hand tools, or better with pneumatic chisels. If the bars have to be cut down, this is done with hack saws or, if available, with oxyacetylene plant. Sometimes the exposed bars are bent, e.g. in the pile cap shown in *Fig. 17*. Any concrete cracked during

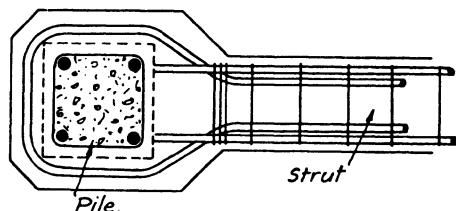


Fig. 50.—Detail of Connection between Pile and Strut.

this process should be cut away before re-concreting. In wharf and viaduct work a strut often has to be secured to a pile at some distance below the top. In this case a short length of the pile concrete has to be cut out, and great care must be taken to pack the new concrete which replaces it close up to the top of the cut. The links which are cut out need not be replaced. The type of joint shown in *Fig. 50* has the advantage that the concrete need not be cut away except for, say, half an inch on the surface.

Cutting off Pile Heads with Explosives.

A method of blowing off the heads of piles which has proved very successful at Auckland Harbour, New Zealand, is as follows. A chase is cut completely around the pile at the level to which the concrete is to be removed. This is effective in preventing the concrete below this level being shattered.

Where the length of concrete to be broken down does not exceed 3 ft. 6 in., a single hole about $1\frac{1}{4}$ in. diameter is drilled down the centre of the pile to within 4 in. of the level to which the concrete is to be removed. A quarter of a $1\frac{1}{8}$ -in. diameter plug (about $\frac{1}{12}$ lb.) of gelignite with a detonator and fuse attached is inserted to the bottom of the hole. The charges are not tamped.

Where the length of concrete to be removed exceeds 3 ft. 6 in., holes are drilled into the sides of the pile at about 3-ft. centres, and a similar charge is inserted in each hole. The charges are fixed and the concrete completely shattered so that it is readily removed from the reinforcement.

CHAPTER IX

CAST-IN-SITU PILES

THE use of concrete piles which, instead of being pre-cast, are made by forming a hole in the ground and filling it with concrete—plain or reinforced—is steadily extending. This class of pile has been used in recent years for some very

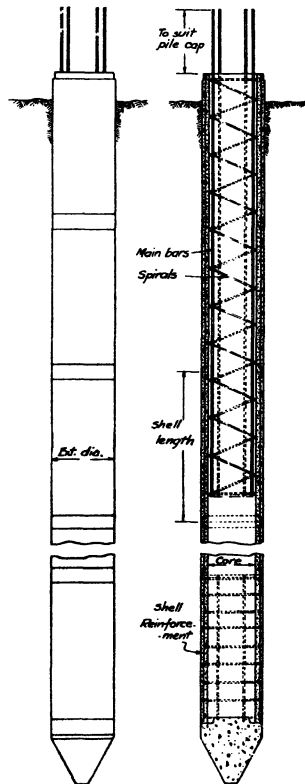


Fig. 51.

important works, and there are many situations where it has decided advantages. Most types of cast-in-situ pile are protected by patent rights or at least are made by firms specialising in one form of pile or another. Brief descriptions of some of these are given in "The Concrete Year Book."

REINFORCED CONCRETE PILING

A cast-in-situ pile necessarily involves making a hole in the ground and this nearly always means that the hole has to be lined with a tube or shell of some sort. In some types the tube is left in the ground and filled with con-

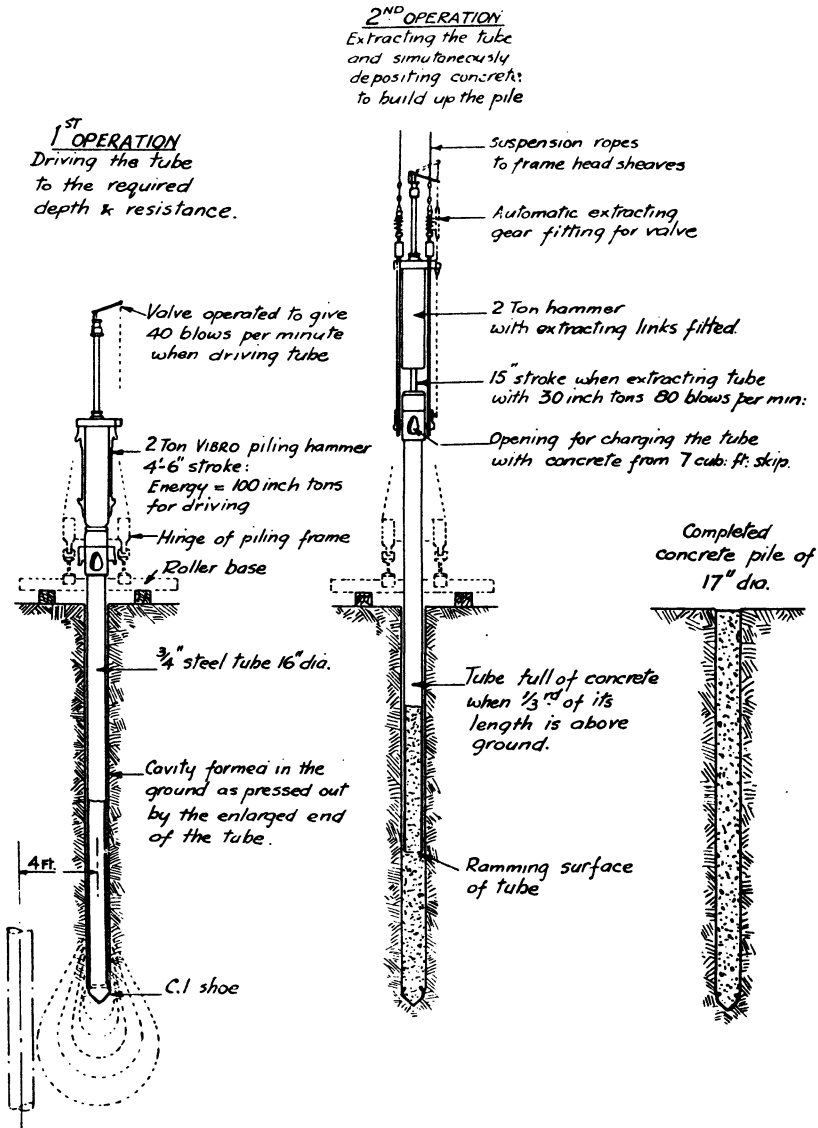


Fig. 52.

crete; in others it is withdrawn as the concrete is being poured into it. In some types the tube is hammered down, and in others the earth is removed from inside the tube by boring, whereupon it descends by its own weight aided perhaps by a very light hammer.

In most types it is possible to introduce reinforcement. This is built up as a cylindrical cage and lowered into the tube after it has been sunk to its final depth, and just before concreting.

In one type of pile where the tube is left in permanently the tube is made of thin steel and tapered. During driving it is lined with a strong steel mandrel which, on being hammered down by a pile driver, carries the tube with it. The mandrel is collapsible and can be withdrawn on completion of the driving, when the tube is filled with concrete. In another system (*Fig. 51*) a parallel cylindrical mandrel is used and on it are threaded reinforced concrete "shells" or short lengths of tube as the mandrel descends under the hammer. The shells remain when the mandrel is withdrawn, and are then filled with concrete.

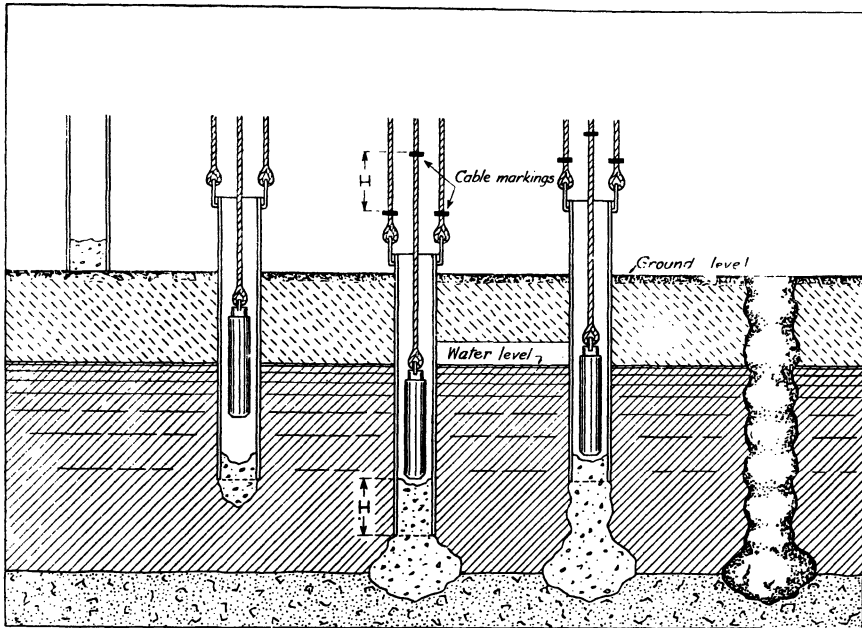


Fig. 53.

As regards types where the tube is withdrawn, in one system (*Fig. 52*) the steel tube is driven down by a pile driver, having been provided with a detachable cast-iron shoe which is left in to form a base to the concrete column when the tube is withdrawn. The tube is driven by a double-acting steam hammer and withdrawn by the same tool hammering upwards with a succession of small jerks, so that the concrete is shaken out of the tube and tamped.

In another system (*Fig. 53*) no shoe is provided, but when the tube is in position ready for driving a small charge of very dry concrete is poured into the tube and consolidated by blows from a cylindrical drop-hammer which works inside the tube. As the blows are continued the friction between the concrete and the tube suffices to drag down the latter. When the required depth is reached further sinking of the tube is prevented by hanging it to the pile-driving

frame, but hammering is continued and the concrete is forced out below the tube to form a bulb, thus increasing the bearing area of the pile. Eventually the hole is filled with concrete consolidated by hammering while the tube is being slowly withdrawn by lifting it from the pile frame.

In a system where the tube is sunk by excavating the core with boring tools, compressed air is used to force out any water which may collect in the tube owing to the absence of a shoe, and also to consolidate the earth at the foot of the tube when sunk, cement grout being injected to aid this. The compressed air also serves to compress the concrete as it is filled in while the tube is being withdrawn. The successive stages in the formation of this class of pile are illustrated in *Figs. 54 to 59*.

Something may be said generally about the relative merits of pre-cast and cast-in-situ piles, and of the varieties of the latter pile at present in use, which may help an engineer in making his choice.

Comparing in-situ piles generally with pre-cast piles it may be said that the former have certain advantages :

(1) They save the delay which occurs while a pre-cast pile is hardening in order to enable it to be driven. This, in these days of rapid-hardening cements, is not perhaps of great importance.

(2) In cases where the driving depth of the pile varies with the nature of the ground they save the labour and waste of cutting off the head of a pile which is too long or the delay caused by lengthening one which is too short.

(3) They prevent the risk of damaging the concrete by heavy hammering.

(4) In many cases, the tube being lighter than the pile, the driving plant can be lighter.

On the other hand, pre-cast piles have certain advantages over in-situ piles :

(1) The concrete is usually of better quality, partly because inspection and tamping are easier and partly because there is no danger of it being damaged by segregation during deposition, or by water or earth getting into the concrete before it has set.

(2) Unlike in-situ piles, they can be driven in water and are perhaps safer in flooded earth.

(3) The reinforcement can easily be carried right down to the foot of the pile.

(4) Generally speaking, pre-cast piles are not protected by patents, and can be freely used by all contractors or building organisations.

Comparing various types of in-situ piles, those where the tube is left in have certain advantages, including

(1) There is less risk of the concrete becoming mixed with earth or water while it is still unset.

(2) The tube has not to be lifted out, which sometimes gives trouble.

(3) There is less risk of the partly-set concrete being damaged by the driving of a neighbouring pile or by earth movement from any cause.

On the other hand the withdrawal of the tube has certain advantages. For example :

(1) The cost of the tube is saved.

(2) The concrete has a rougher surface and therefore perhaps a better frictional hold on the ground. (In settling ground this may not be advantageous.)

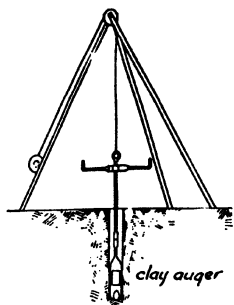


Fig. 54.

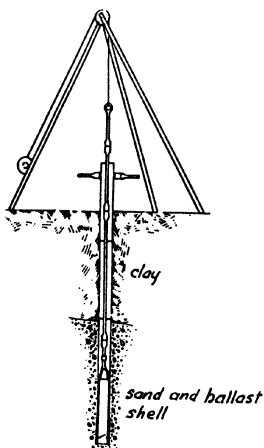


Fig. 55.

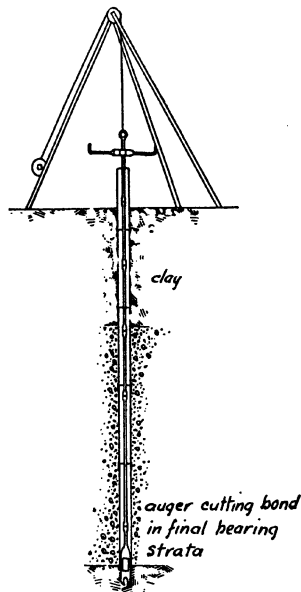


Fig. 56.

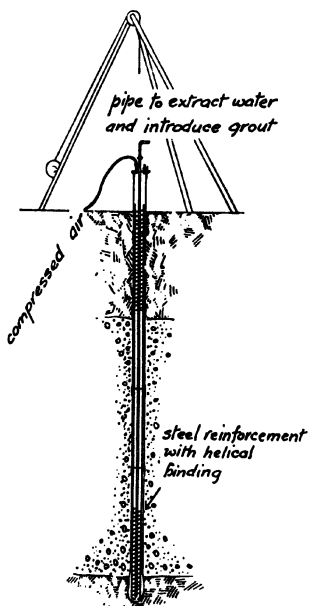


Fig. 57.

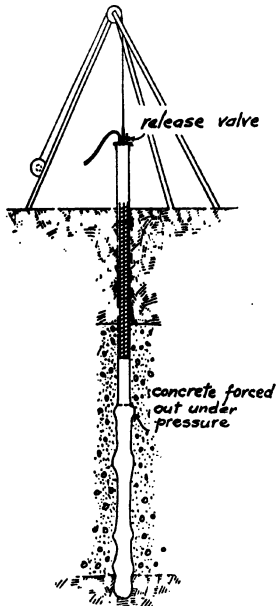


Fig. 58.

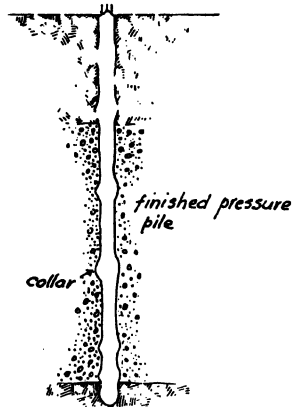


Fig. 59.

Piles which are hammered down have certain advantages over those which are sunk by boring. These include :

(1) The hammering tightens up the ground and this increases its bearing power in most cases (though not in all).

(2) The record of set and hammer blow gives a useful indication of the load the pile will bear.

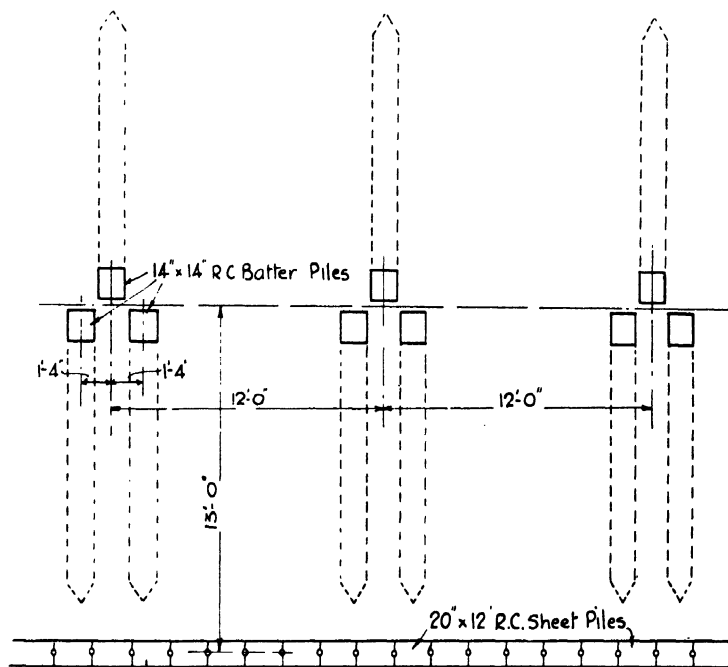
On the other hand, a pile driven by boring is to be preferred in positions where hammering will do damage as, for instance, when close to a frail building.

CHAPTER X

DESCRIPTIONS OF PILING WORKS

Hamworthy Wharf.

A good example of the type of design referred to on page 37 is the wharf erected at Hamworthy, Dorset, for the Poole Harbour Commissioners from the designs of Sir Alexander Gibb & Partners. It is shown by plan and cross section, *Figs. 60*



PART PLAN SHOWING PILING

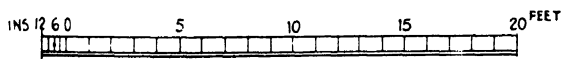


Fig. 60.—Hamworthy Wharf.

and 61, from which it will be seen that the wharf is designed for a depth of 20 ft. at L.W.O.S.T. and 26 ft. 6 in. at H.W.O.S.T. It is built entirely of reinforced concrete, the face of the wharf below mean tide level consisting of a continuous

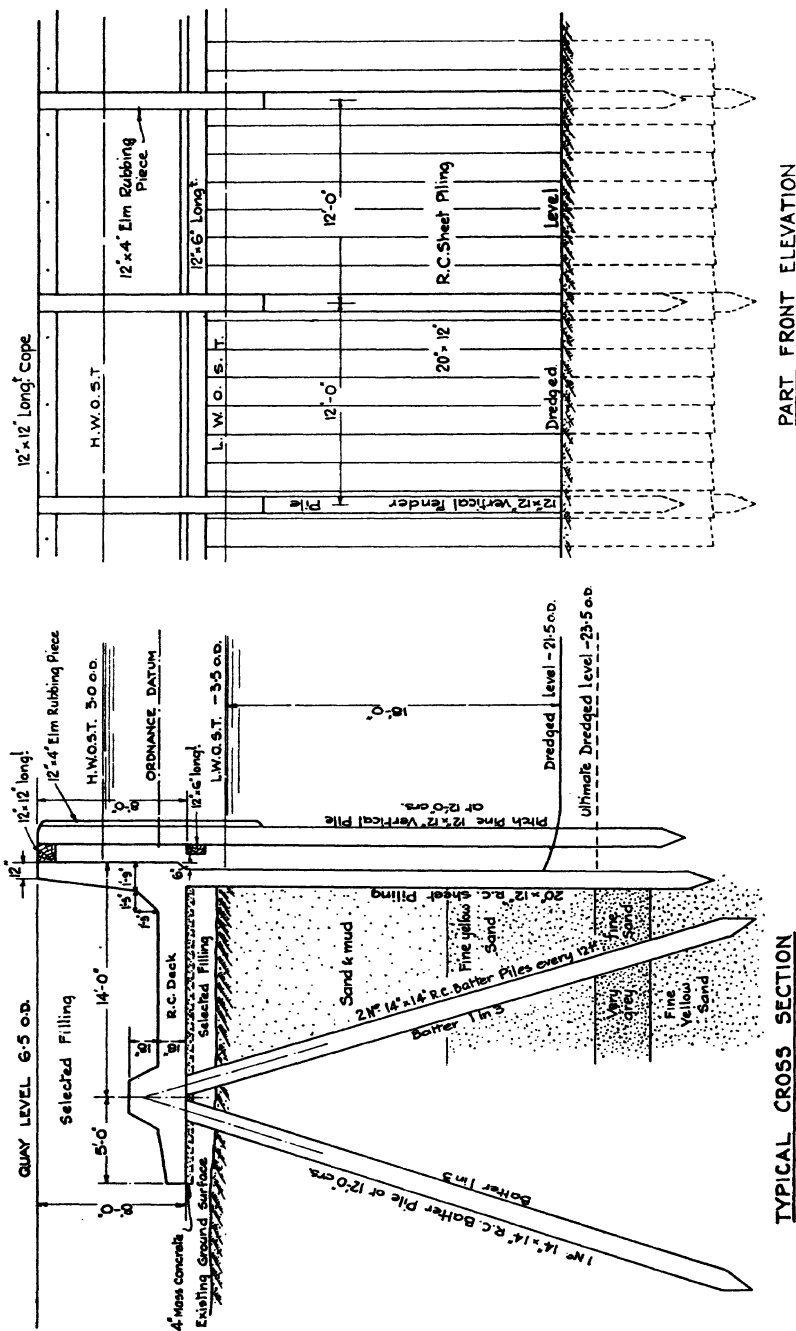


Fig. 61.—Hamworthy Wharf.

row of piles each 20 in. wide and 12 in. thick, and above this level a wall 1 ft. 9 in. to 1 ft. thick. The tops of the piles and the base of the wall are secured to a continuous horizontal slab 19 ft. wide and mostly 1 ft. 6 in. thick. This slab is anchored at intervals of 12 ft. to groups of three 14-in. piles battered at 1 in 3, two sloping towards the face of the wharf and one away from it. The face of the wharf is protected from abrasion by 12-in. timber piles with 4-in. rubbing pieces spaced 12 ft. apart. Details of the deck and the main beams are shown in *Figs. 62 and 63*.

It was estimated that the horizontal pull on the slab due to the earth backing might amount to 30 tons per 12-ft. bay. The vertical load on the group of three anchor piles due to the earth above them was estimated at 72 tons. A force polygon showed that this combination would impose a thrust of 42 tons on each of the two forward batter piles and a pull of 9 tons on the after pile. A live load (due to traffic on the quay and estimated at 36 tons) would increase

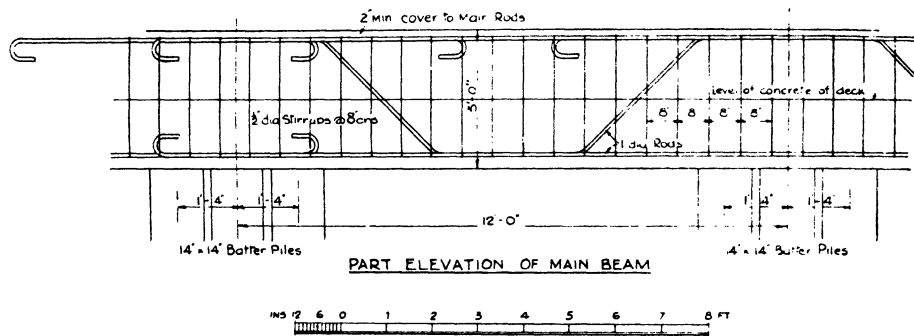


Fig. 63.—Hamworthy Wharf.

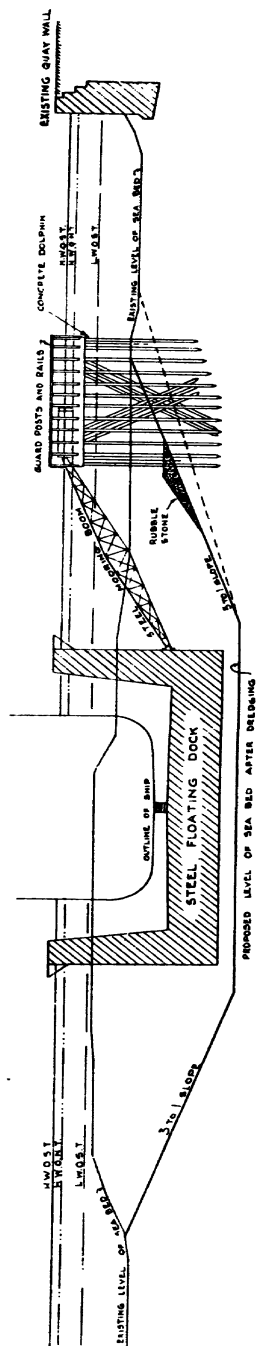
the thrust on each of the two forward piles to 52 tons and would also change the force on one after pile from a pull to a thrust of 10 tons.

It will be seen from the drawing that the soil behind the wharf consisted of fine sand overlaid with silt and sand whose surface was at about low-tide level. After the wharf was built the space behind was filled up to quay level with selected material and the ground in front dredged away to form the berth. On account of the compact nature of the fine sand into which the piles had to be driven, a water jet had to be used.

Floating Dock, Southampton.

The floating dock at Southampton is moored transversely by means of four hinged steel booms to four dolphins. It was estimated that with a ship in dock, and with a wind-pressure of 20 lb. per square foot, the push or pull on the dolphins might amount to 1,000 tons, or 250 tons per dolphin. Moreover, as an oblique wind would cause the dock to range forward or aft to a certain extent, it was estimated that a side thrust or pull might be produced on the dolphins amounting to 72 tons or 18 tons per dolphin.

The dolphins, which were constructed by Messrs. A. Jackaman & Son, Ltd., are shown in *Figs. 64 and 65*. Each consists of a massive reinforced concrete slab, or cap-block, resting on ninety-four reinforced concrete piles, ranging in



CROSS - SECTION
(DOCK SUBMERGED AT H.W.O.N.T.)

Fig. 64.—Dolphin at the Floating Dock, Southampton.

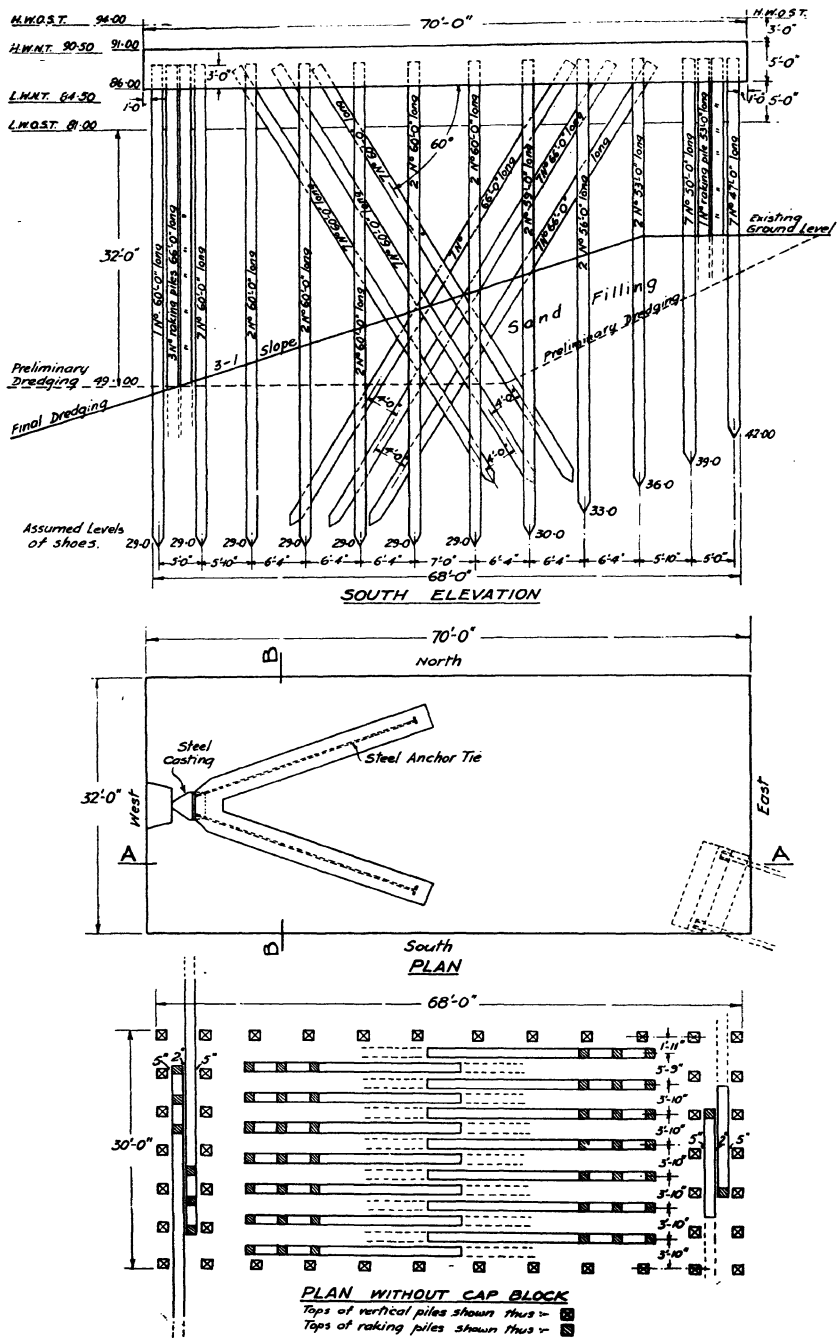


Fig. 65.—Plans and Elevation of Dolphins at Southampton Docks.

length from 47 ft. to 66 ft., and all 16 in. square. Of these piles forty-four are vertical, the remaining fifty being sloped at 30 deg. with the vertical in all four directions. The cap-block is 70 ft. long, 32 ft. wide, and 5 ft. thick. Above the block is a reinforced concrete wall, V-shape in plan, containing a pair of steel anchor ties which are attached to a heavy steel casting forming the socket of the cruciform pin by which the steel boom is fastened to the dolphin.

The concrete was made with a good clean gravel from Langston Harbour. For the piles, the sand and stones were separated by screening and all stones above $\frac{3}{4}$ in. were rejected. The concrete was then mixed in the proportions of 1:1 $\frac{1}{2}$:2 $\frac{1}{2}$. For the cap-block, where the reinforcement was more widely spaced, the gravel was used in its natural state, and the concrete was mixed in the proportions of 1:4. The piles were reinforced with four 1 $\frac{3}{8}$ -in. steel rods and with $\frac{3}{16}$ -in. links. The reinforcement of the cap-blocks is illustrated in *Fig. 66*.

The dimensions of the dolphins were chosen so that the thrust on the piles should in no case exceed 30 tons and so that the pull should not exceed 6 tons. The stresses in the concrete and steel were limited to 600 lb. and 16,000 lb. per square inch respectively.

The wind pressure of 250 tons conveyed by the booms to each dolphin is resisted by the weight of the structure combined with the thrust induced in the raking piles. The thrust of 500 tons is equally shared by the 21 raking piles. Hence the thrust on each is $\frac{500}{21} = 23.8$ tons. Moreover, certain thrusts

and pulls are induced in the vertical piles, and bending stresses are induced in the cap block. The method of estimating these forces was as follows.

A pull on the dolphin of 250 tons at *A* (*Fig. 67*) may be resolved into a thrust of 500 tons on the raking piles at *B*, and an upward vertical force of 433 tons at *C*. The vertical force of 433 tons must be resisted by the weight of the cap-block and the pulls or pushes of the vertical piles beneath it. At low tide, when the cap-block is exposed, its weight is 750 tons, and its centre of action is at *D*. The two unequal parallel forces *C* and *D* (*Fig. 68*) may be replaced by a downward force at *D* of magnitude $W = (750 - 433) = 317$ tons and a clockwise couple whose moment is $433 \times 22.5 = 9,675$ ft.-tons. The former will be equally shared by the 44 vertical piles, and will produce a push of $\frac{317}{44} = 7.2$ tons on each pile. The couple will be resisted by pushes and pulls in the group of 44 vertical piles.

The push or pull *P* exerted on any pile is obtained from a modification of the well-known formula $p = \frac{W}{A} \pm \frac{My}{I}$ used in finding the stresses in eccentrically-loaded footings. The required formula is

$$P_x = \frac{W}{n} \pm \frac{Mx}{\sum x^2}$$

where P_x = push (or pull) on pile at distance x from point *D*,
 W = total force at *D* (tons),
 n = number of vertical piles,
 M = bending moment (ft.-tons), and
 x = distance of pile from *D*.

Here $W = 317$ tons,
 $n = 44$,
 $M = 9,675$ ft.-tons,
 and Σx^2 is found as follows :

Number of piles in row	Value of x for this row	Number $\times x^2$
	ft. in.	
2×7	33 4	$14 \times 1111 = 26,796$
2×7	28 4	$14 \times 803 = 2,024$
2×2	22 6	$4 \times 506 = 1,044$
2×2	16 2	$4 \times 261 = 388$
2×2	9 10	$4 \times 97 = 48$
2×2	3 6	

Total = $\Sigma x^2 = 30,300$ for 44 piles.

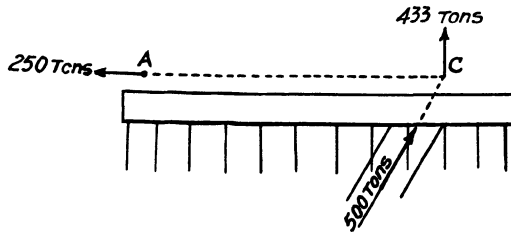


Fig. 67.

Then to find the push (or pull) on the outermost pile (where $x = 33$ ft. 4 in.) substituting

$$\begin{aligned}
 P_{x=33'4"} &= \frac{317}{44} \pm \frac{9675 \times 33.33}{30,300} \\
 &= 7.2 \pm 10.6 \\
 &= 17.8 \text{ tons} \\
 &\text{or } -3.4 \text{ tons.}
 \end{aligned}$$

That is, each pile at the left end will sustain a push of 17.8 tons and each pile at the right end a pull of 3.4 tons.

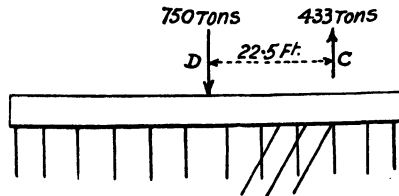


Fig. 68.

At high water the cap-block is submerged, and its weight is consequently reduced by flotation. The pushes and pulls on the various vertical piles are consequently different, but are calculated in the same way.

In order to calculate the bending moment in the cap-block, it was considered as a beam acted upon by

- (a) the weight of the block ;
- (b) the various pushes or pulls in the vertical piles obtained as described ;
and
- (c) the vertical components of the pushes in the raking piles.

This gave a maximum bending-moment of 56 ft.-tons per foot width of block.

Deep-Water Quay at Havre.

Figs. 69 and 70 illustrate a clever design for a wharf to accommodate the largest vessels at the Port of Havre, France, in which reinforced concrete piling plays a conspicuous part. As will be seen from the cross section it consists

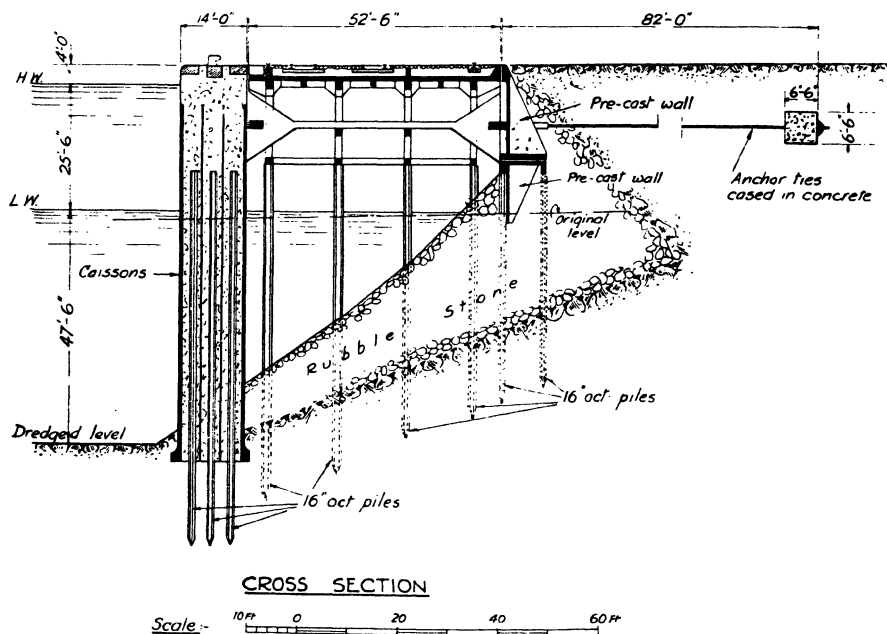
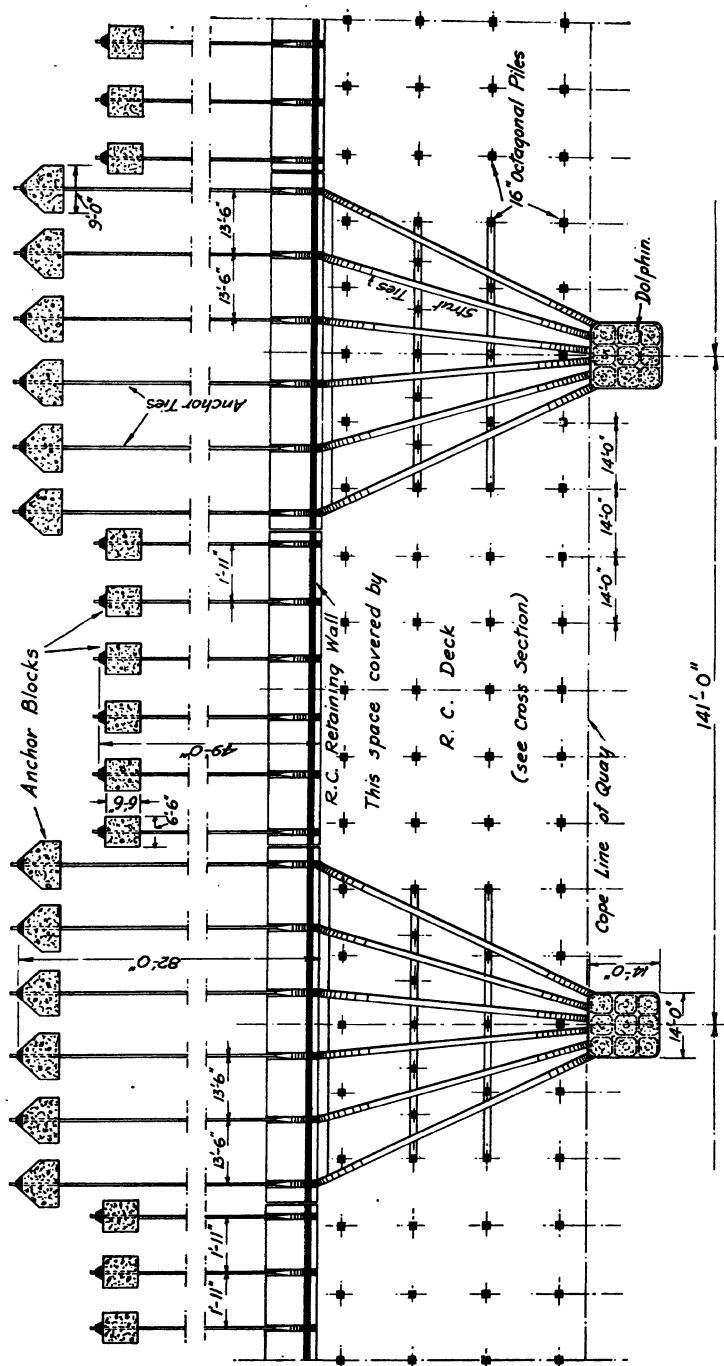


Fig. 69.—Deep-Water Quay at Havre.

mainly of a deckway 52 ft. 6 in. wide supported on four rows of piles spaced about 14 ft. apart. This is of a comparatively light character as it has only to take the weight of the cranes and traffic and is relieved by other parts of the structure from all lateral loads such as the thrust from the earth backing and the blows from berthing ships. The dredged slope of sand underneath the wharf is steepened by casting out rubble stone (broken chalk) which stands at about 1 to 1. The slope is surmounted by a retaining wall of reinforced concrete formed in precast lengths and resting on two rows of piles and tied to anchor blocks buried in the sand. In front of the quay at intervals of 140 ft. are high cellular caissons of reinforced concrete secured to the ground by piles and laterally by



PLAN
 (BELOW DECK)

Fig. 70.—Deep-Water Quay at Havre.

strut-ties to other anchor blocks buried in the sand. The caissons and strut-ties take the push or pull of ships alongside the quay, and are designed for a push of 500 tons and a pull of 100 tons.

Groynes at Bournemouth.

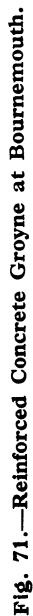
Fig. 71 illustrates a type of groyne constructed in 1933 by Mr. F. P. Dolamore, then Borough Engineer, to protect the foreshore at Bournemouth. It consists of a continuous row of 12-in. by 6-in. reinforced concrete piles crossed by lines of 9-in. by 9-in. piles of the same material. The transverse rows are about 8 ft. long and are spaced at intervals of 7 ft. and consist for the most part of two piles only, but every third set (21-ft. intervals) consists of six piles in a row. These transverse lines of piles serve a double purpose. They act as wave breakers, and thus minimise scouring action along both faces of the groyne, and they also acted as a "viaduct" to support the pile-driving frame during construction. The frame travelled on temporary rails supported on the outside transverse piles. It was 21 ft. long and could overhang by 7 ft. the latest pair of piles driven so as to be in position to drive the next pair. With this arrangement all piles could be driven and the groyne carried out into fairly deep water (which is, of course, most important to make a groyne effective). Moreover the frame could easily be drawn back shorewards in rough weather. The piles were light, and driving (into sand) was aided with a water jet so that a 15-cwt. hammer sufficed to drive them. After driving, the piles were stiffened as shown in the drawing by connecting their heads with longitudinal and transverse caps.

Pile Yard at Southampton Docks.

The piles for the Passenger and Cargo Sheds Nos. 105 to 108 at Southampton Docks were 400 in number and were reinforced concrete piles of 15 in. by 15 in. section, and ranging in length from 47 ft. to 55 ft. The concrete was a 1 : 1½ : 2½ mix, and the piles were reinforced with four 1½-in. rods with ¼-in. links and with steel diagonal forks; the piles were designed to carry a maximum load of 37 tons. They were provided with cross-holes (lined with 1½-in. tubes) for lifting the piles (at two points), and for inserting toggle bolts to secure them to the frames while driving.

The pile yard was sited on the quay space at the extreme west end of the sheds, and was laid out as shown in *Fig. 72*. The accommodation included an area for building up the steel skeletons, another on which the pile moulds were laid down and where concreting took place, and another where the finished piles were stacked while maturing.

The building of the steel skeletons is shown in *Fig. 75*, from which it will be seen that they were erected on stools about 4 ft. high and spaced about 15 ft. apart and surmounted with light steel frames from which the main bars were suspended in correct relative positions so that links and forks could be conveniently placed on them. When completed the skeletons were lifted by a derrick crane (with the aid of a spreader made of two railway rails, shown in the photograph, which enabled the weight to be taken at three points) and placed in the moulds ready for concreting. The gang consisted usually of five men. The main bars were bent with power-driven machines and the links by hand.



REINFORCED CONCRETE PILING

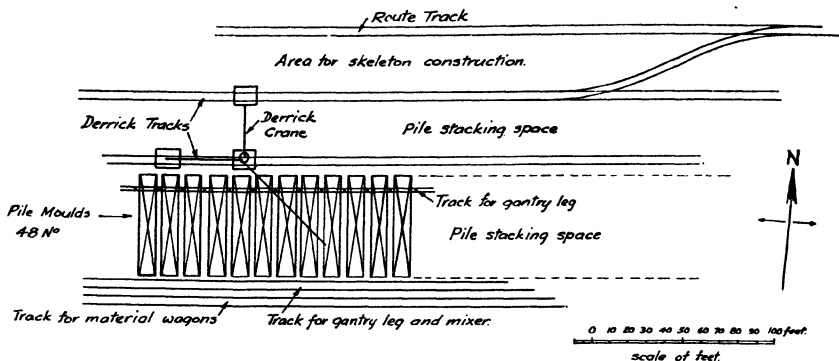


Fig. 72.—General Arrangement of Pile Yard at Southampton Docks.

The pile moulds are shown in *Figs. 72 and 73*. There were 48 in all, placed side by side in a north-and-south direction with standard-gauge rail tracks running east and west on both sides of them. The two tracks south of the moulds served to bring up wagons containing cement and aggregate. One track served also

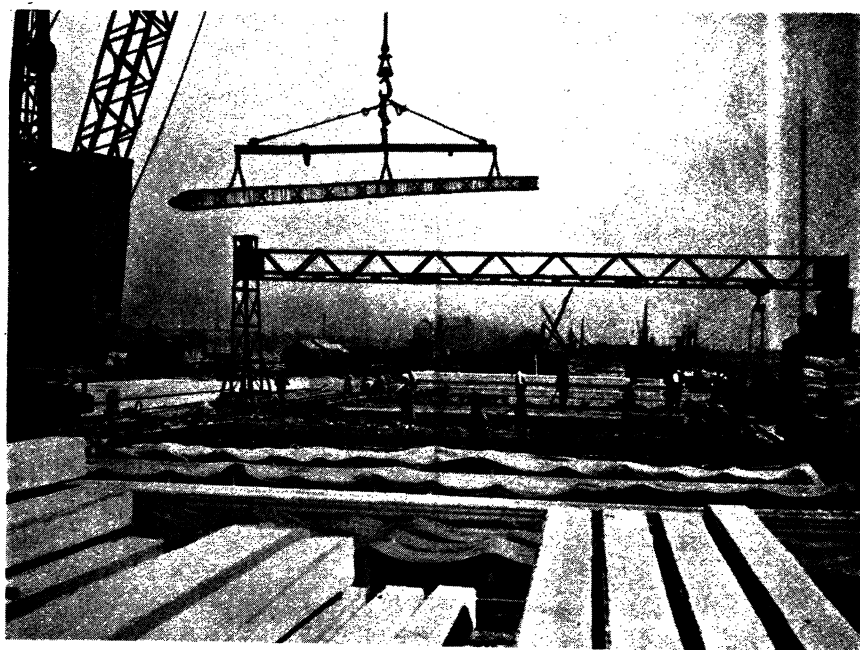


Fig. 73.—Pile Yard for Sheds Nos. 105–108, Southampton Docks.

to carry one leg of a travelling gantry which supported a runway on which skips of mixed concrete ran over each pile in turn from the mixer, which was mounted on that leg. The other leg of the gantry travelled on a narrow-gauge rail track which ran over the moulds. Two rail tracks north of the moulds, spaced at

25-ft. centres, carried a steam travelling derrick crane of 80-ft. radius, which commanded not only the moulds, but also the reinforcement assembling area and the pile-stacking area to the east and north of the former. Thus the derrick was able to lift the steel skeletons and place them in position for concreting, and later to lift the finished piles and stack them and thus release the moulds for other piles. It also loaded piles on to the wagons when ready for driving. Another track north of the derrick tracks served as a route road, on which piles were taken away to be driven, and for general purposes.

The 48 moulds were designed for an output of 72 piles per week. They are shown in detail by cross section in *Fig. 74*. The ground on which they were placed was a gravel embankment which had become fairly well consolidated. On it were laid 9-in. by 5-in. sleepers at 6-ft. intervals running north and south, and covered by 4-in. by 3-in. bearers at 3-ft. intervals running east and west. To these were spiked the 15-in. by 1½-in. mould bases, spaced so as to give a clear distance of 1 ft. 2 in. between the sides of the moulds. This distance was

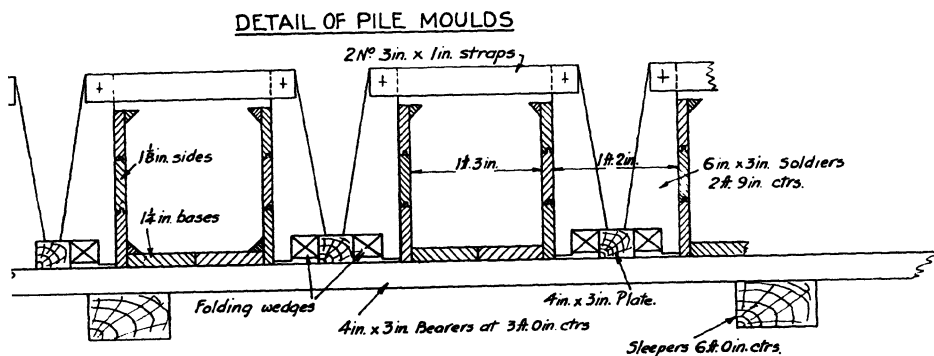


Fig. 74.—Pile Moulds at Southampton Docks.

increased at every fourth interval so as to accommodate a prop for the narrow-gauge rail track for the northern gantry leg, which, as mentioned previously, ran over the piles. The mould sides were of 1½-in. timber in 11-ft. lengths, and were stiffened by pairs of 6-in. by 3-in. soldiers, each pair being placed 2 ft. 9 in. apart, and connected above the moulds by 3-in. by 1-in. timber straps. The bases of the soldiers were rebated so that they could be wedged to a 4-in. by 3-in. plate running parallel to and between the mould beds and fixed to the bearers. This arrangement kept the sides of the moulds firm while being concreted but allowed them to be easily freed by striking out the wedges when the concrete had set.

The mixer shown in *Fig. 75*, which, as mentioned previously, was mounted on the south leg of the travelling gantry, was a petrol-driven ¼-cb. yd. batch machine. It was provided with a feeding hopper into which coarse and fine aggregate were shovelled directly from a railway wagon standing alongside. The hopper was marked to show the correct volume required to correspond with 1½-cwt. cement which was fed in at the same time. The water was admitted from an overhead tank, the quantity being regulated to produce the required

slump, which was laid down to be 6 in. on a 12-in. cone. When mixed, the concrete was discharged into a skip provided with a small spout closed by a door, which was released after the skip had been pushed along the gantry runway to that part of the mould where concrete was needed. This arrangement of plant was economical and the estimated output of 72 piles per week was very nearly maintained.

The sides of the moulds were struck after 24 hours and each pile was left to mature for at least 90 hours, during which period the piles were kept wet. It was then lifted and stacked until it was at least 21 days old and then loaded and sent away for driving.

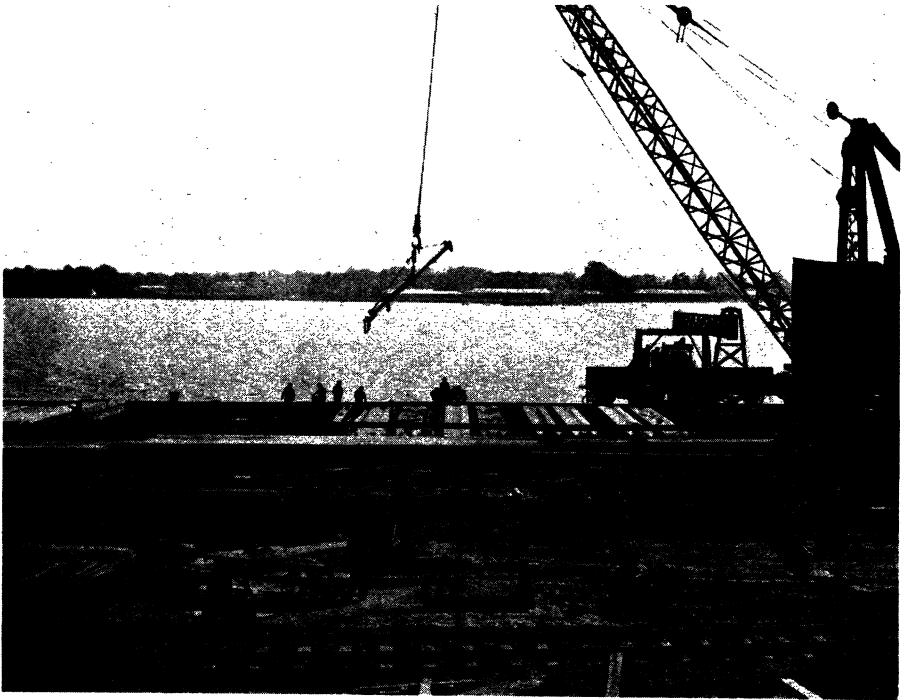


Fig. 75.—Pile Yard for Sheds Nos. 105–108, Southampton Docks.

The gang attending to the moulds consisted generally of 4 men (2 carpenters and 2 labourers) and the concreting gang of 12 men who were disposed as follows : 1 ganger, 1 mixer driver, 3 feeding mixer with aggregate, 1 feeding mixer with cement, 2 moving skip, and 4 ramming, etc. This gang also attended the derrick when lifting piles. Their output averaged about 68 piles or, say, 200 cb. yd. per week. There was no night shift but a good deal of overtime.

The piles were driven by means of two frames with 3-ton semi-automatic hammers. Each frame was served by a 10-ton derrick crane which placed the frame, lifted the piles from the wagons which brought them from the yard and offered them to the frame, and also lifted the hammer into the guides. In each

case steam was supplied from a portable loco-type boiler mounted on a flat wagon attached to the derrick crane.

The pile-driving gang worked day and night shifts, and consisted of : 1 charge hand, 1 man driving the derrick, 1 man attending the boiler, and 2 men attending the frame, with an extra man at night. The output was about $3\frac{1}{2}$ piles driven per frame per shift and, with both frames working, about 66 piles per week. The 400 piles were in fact driven in eight weeks. As they consisted largely of single piles or pairs spaced 49 ft. apart, a large proportion of the time was occupied in moving frames and derricks. The work was carried out for the Southern Railway Company by Sir Robert McAlpine & Sons (London), Ltd., who devised the lay-out here described.

Pile Yard at Manchester Ship Canal.

The piles for the transit sheds at the Manchester Ship Canal were of varying lengths up to 45 ft. and octagonal in section, 16 in. across the flats. The concrete was a 1 : $1\frac{1}{2}$: 3 mix and the piles were reinforced with 8 longitudinal bars surrounded by spirals.

The pile yard was a long rectangular site shown in *Fig. 76*. The area on which the steel skeletons were built is seen at the near end and the area containing the pile moulds beyond it. About half-way along this area on the left is seen the concrete mixer with dumps of coarse and fine aggregate on this side



Fig. 76.—Pile Yard at Manchester Ship Canal.

of and beyond it. The photograph also shows a 75-ft. radius derrick crane which served to unload materials, to pick up the steel skeletons and place them in the moulds, and to pick up the finished piles and load them for stacking or driving. The various rail tracks can also be seen. Five lines of narrow-gauge railway ran along the length of the yard, each feeding three pile moulds on each side. These served to bring skeletons from the steel area and concrete from the mixer, to which they were connected by means of another narrow-gauge railway running across the site and equipped with turntables. Other short



Fig. 77.—Building Pile Skeletons, Manchester Ship Canal.

lengths of narrow-gauge railway served to bring the aggregate from the dumps to the mixer. The standard-gauge railways seen on each side of the yard served to bring up materials and to take away the piles. The crane itself ran on two standard tracks.

The building of the steel skeletons is shown in *Fig. 77*. The longitudinal bars with their spreaders were suspended from a series of "gallows" by ropes by which the skeletons could be adjusted to any required height, and after they had been surrounded by the spirals they could be lowered to the ground or on to bogie wagons, by means of which they were conveyed to the pile moulds.

Generally the steel gang comprised 2 men straightening and bending bars, 2 men making spirals, and 6 or more men assembling.

The pile moulds are shown in *Fig. 78* and were formed by laying down continuous lines of 2-in. boards resting on 5-in. by 2-in. sleepers spaced 5 ft. apart. These boards formed the beds of the moulds, the three side faces being formed with mild steel shutters in 10-ft. lengths. These were placed in position after a skeleton had been laid down, and were pressed tight against the beds by means of loose planks placed between the shutters and chocks nailed to the

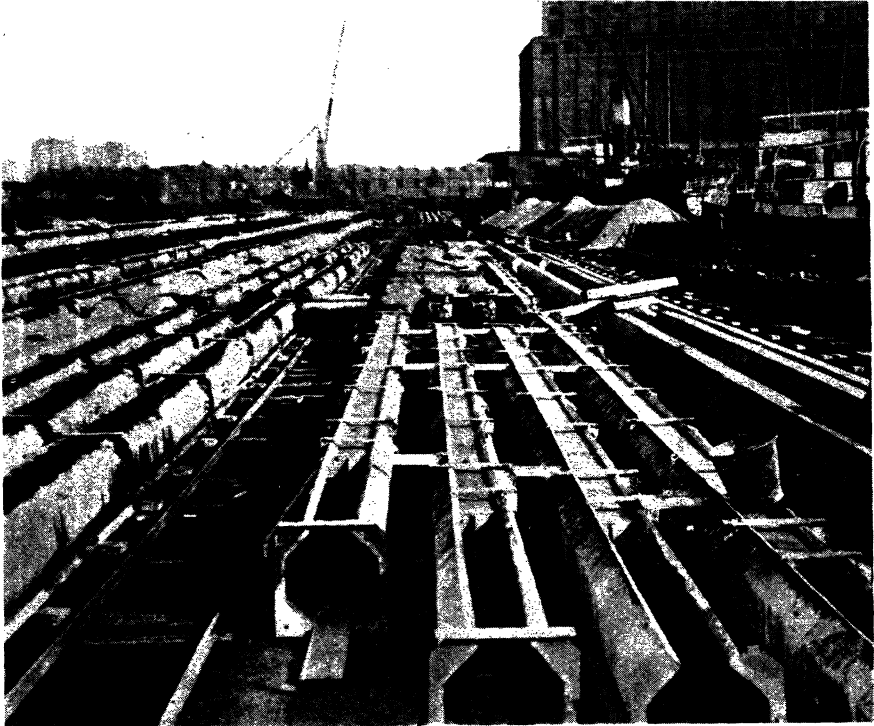


Fig. 78.—Pile Moulds, Manchester Ship Canal.

sleepers, wedges being inserted as required. The steel shutters were further prevented from spreading by means of the bars above them placed 5 ft. apart, and which can be seen in the photograph. A special form containing the shoe was placed between the steel shutters at the point of the pile, and the head was formed by an octagonal timber shutter. It will be observed that the tops of the steel shutters were flanged to facilitate pouring the concrete, and that sufficient space was left between the moulds to facilitate striking the shutters from any pile without disturbing its neighbour. To ensure the correct placing of the steel the skeletons were hung by wires from the top of the steel shutters.

The mixer was an electrically-driven rotary batch mixer of half cubic yard

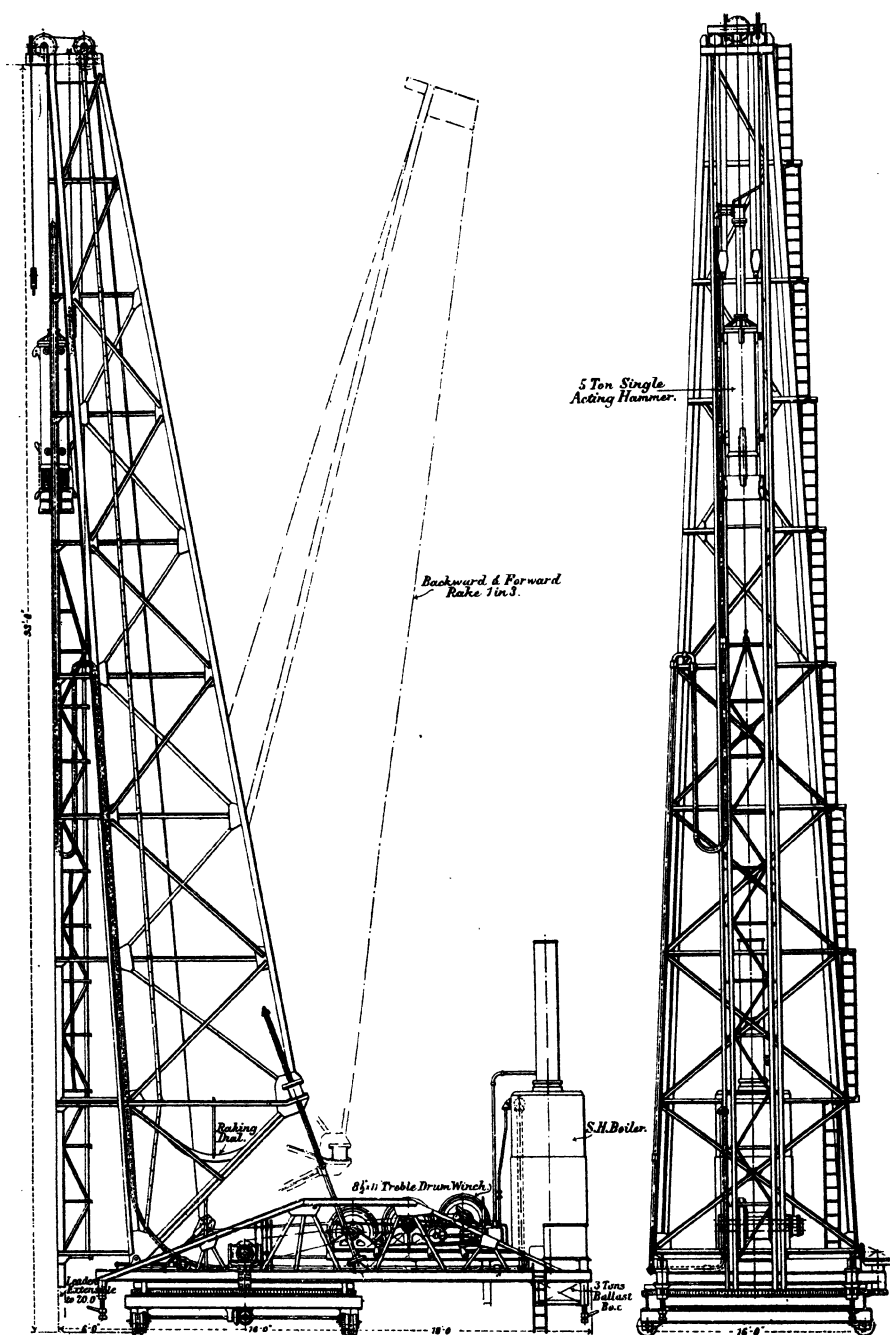


Fig. 79.—Pile-driving Machine 93 ft. high.

capacity. It discharged into a rectangular box 4 ft. by 3 ft. by 16 in. resting on a bogie wagon which travelled on the narrow-gauge railway by which it was transported to the moulds. There one side of the box was removed and the concrete shovelled out directly into the nearest mould, or over a light staging to the second or third.

Each batch of concrete consisted of 305 lb. of cement ($1\frac{1}{2}$ bags), 5.08 cb. ft. of sand, and 10.17 cb. ft. of coarse aggregate, yielding about $11\frac{1}{2}$ cb. ft. of wet concrete. The concreting gang consisted usually of 1 ganger, 2 men on the mixer, 2 on cement, 2 on coarse aggregate, 2 on sand, 2 on transporting, 6 unloading and ramming, 3 fixing forms (2 carpenters and 1 labourer, and 2 fixing pile shoes, hanging skeletons, striking and cleaning moulds, etc. This gang could concrete nine piles 45 ft. long by 16 in. (octagonal) in a nine-hour day, representing an output of about $21\frac{1}{2}$ cb. yd. of concrete. All materials were unloaded by small piecework gangs. The work was carried out for the Manchester Ship Canal Co. by Sir William Arrol & Co., Ltd.

Large Pile-Driving Machine.

Fig. 79 shows front and side elevations of a large pile-driving machine built for the South Australian Harbour Board. The frame is 93 ft. high and is designed for driving piles 70 ft. long and 24 in. square and weighing 15 tons. The piles can be driven vertical or with a batter up to 1 in 3 forwards or backwards. The tilting is performed by power-operated screws shown in the side elevation. The frame is capable of rotating on its undercarriage through a complete circle and is stable under all conditions without the use of guy ropes.

The undercarriage is mounted on four cast-steel travelling wheels at 16-ft. centres, and is designed to run on the road or on flat-bottom rails. All four wheels are geared for power drive. The swivelling carriage is provided with a jack at each corner. Normally the jacks serve to steady the plant when working, but they can also be employed to lift the swivelling carriage independently of the undercarriage and so permit the latter to be slewed on its track.

The winch has three drums driven by two steam cylinders of $8\frac{1}{2}$ in. by 12 in. bore and stroke. The drums are 14 in. diameter and driven through friction clutches. The operating levers are placed just ahead of the front drum, where the driver can see and control the various operations of raking, travelling, slewing, and traversing the frame to clear a driven pile.

The hammer is of the single-acting type having a 5-ton falling cylinder with a maximum stroke of 6 ft. The action is semi-automatic, but the stroke is adjustable in steps of 1 ft. from zero to the maximum.

The boiler is of the Spencer Hopwood type with a working pressure of 120 lb. per square inch, and an evaporative capacity of 2,050 lb. per hour with natural draught, and 2,650 lb. per hour with forced draught. Excluding the pile and ballast the whole plant weighs 71 tons.

APPENDIX I

Properties of certain Square and Octagonal Reinforced Concrete Sections

THE first five tables give the safe short-column loads and safe moments of resistance of 12-in., 14-in., 15-in., 16-in., and 18-in. square piles with 1½-in. cover on the main bars and made from 1:1:2, 1:1½:3, and 1:2:4 High-grade concrete.

The short column loads are based on a permissible compressive stress on the steel of 13,500 lb. per square inch and on the concrete as follows:

Concrete proportions							Compressive stress
1:1:2	.	.	.	.	.	.	1000 lb. per square inch.
1:1½:3	.	.	.	.	.	.	880 " " "
1:2:4	.	.	.	.	.	.	760 " " "

The safe moments of resistance are based on a permissible tensile stress in the steel of 18,000 lb. per square inch and a permissible compressive stress in the concrete as follows:

Concrete proportions							Compressive stress
1:1:2	.	.	.	.	.	.	1250 lb. per square inch.
1:1½:3	.	.	.	.	.	.	1100 " " "
1:2:4	.	.	.	.	.	.	950 " " "

The depth of the neutral axis and the moment of inertia are included to assist those designing piles in which other proportions are adopted for the concrete.

12-IN. BY 12-IN. PILES.

(Cross-sectional area = 144 sq. in. Weight per foot run = 144 lb.)

Reinforcement: Mild steel, B.S.S. No. 15	Short-column load			Depth of neutral axis (in.)	Moment of inertia (in. ⁴)	Safe moment of resistance (in.-lb.)		
	1:1:2	1:1½:3	1:2:4			1:1:2	1:1½:3	1:2:4
Four 1-in. bars	183,270 lb. (82 tons)	166,370 lb. (74 tons)	149,460 lb. (67 tons)	4.01	1192	239,040	239,040	239,040
Four 1¼-in. bars	205,360 lb. (92 tons)	188,670 lb. (84 tons)	171,980 lb. (77 tons)	4.46	1622	359,250	359,250	345,590
Four 1½-in. bars	232,360 lb. (104 tons)	215,930 lb. (96 tons)	199,490 lb. (89 tons)	4.79	2063	498,930	473,970	409,340

REINFORCED CONCRETE PILING

14-IN. BY 14-IN. PILES.

(Cross-sectional area = 196 sq. in. Weight per foot run = 196 lb.)

Reinforcement: Mild steel, B.S.S. No. 15	Short-column load			Depth of neutral axis (in.)	Moment of inertia (in. ⁴)	Safe moment of resistance (in.-lb.)		
	1:1:2	1:1½:3	1:2:4			1:1:2	1:1½:3	1:2:4
Four 1-in. bars	235,270 lb. (105 tons)	212,130 lb. (95 tons)	188,980 lb. (84 tons)	4.31	1884	294,160	294,160	294,160
Four 1½-in. bars	257,360 lb. (115 tons)	234,430 lb. (105 tons)	211,500 lb. (94 tons)	4.85	2604	475,080	475,080	475,080
Four 1½-in. bars	284,360 lb. (127 tons)	261,690 lb. (117 tons)	239,010 lb. (107 tons)	5.27	3360	622,100	622,100	605,990

15-IN. BY 15-IN. PILES.

(Cross-sectional area = 225 sq. in. Weight per foot run = 225 lb.)

Reinforcement: Mild steel, B.S.S. No. 15	Short-column load			Depth of neutral axis (in.)	Moment of inertia (in. ⁴)	Safe moment of resistance (in.-lb.)		
	1:1:2	1:1½:3	1:2:4			1:1:2	1:1½:3	1:2:4
Four 1-in. bars	264,270 lb. (118 tons)	237,650 lb. (106 tons)	211,020 lb. (94 tons)	4.44	2295	321,810	321,810	321,810
Four 1½-in. bars	286,360 lb. (128 tons)	259,950 lb. (116 tons)	233,540 lb. (104 tons)	5.03	3155	482,310	482,310	482,310
Four 1½-in. bars	313,360 lb. (140 tons)	287,210 lb. (128 tons)	261,050 lb. (117 tons)	5.48	4141	683,700	683,700	683,700

16-IN. BY 16-IN. PILES.

(Cross-sectional area = 256 sq. in. Weight per foot run = 256 lb.)

Reinforcement: Mild steel, B.S.S. No. 15	Short-column load			Depth of neutral axis (in.)	Moment of inertia (in. ⁴)	Safe moment of resistance (in.-lb.)		
	1:1:2	1:1½:3	1:2:4			1:1:2	1:1½:3	1:2:4
Four 1-in. bars	295,270 lb. (132 tons)	264,930 lb. (118 tons)	234,580 lb. (104 tons)	4.56	2750	349,440	349,440	349,440
Four 1½-in. bars	317,360 lb. (142 tons)	287,230 lb. (128 tons)	257,100 lb. (115 tons)	5.18	3845	530,890	530,890	530,890
Four 1½-in. bars	344,460 lb. (154 tons)	314,490 lb. (140 tons)	284,610 lb. (127 tons)	5.68	5011	745,100	745,100	745,100

18-IN. BY 18-IN. PILES.

(Cross-sectional area = 324 sq. in. Weight per foot run = 324 lb.)

Reinforcement: Mild steel, B.S.S. No. 15	Short-column load			Depth of neutral axis (in.)	Moment of inertia (in. ⁴)	Safe moment of resistance (in.-lb.)		
	1:1:2	1:1½:3	1:2:4			1:1:2	1:1½:3	1:2:4
Four 1-in. bars	363,270 lb. (162 tons)	324,770 lb. (145 tons)	286,260 lb. (128 tons)	4.76	3791	404,810	404,810	404,810
Four 1½-in. bars	385,360 lb. (173 tons)	347,070 lb. (155 tons)	308,780 lb. (138 tons)	5.46	5352	616,860	616,860	616,860
Four 1½-in. bars	412,360 lb. (185 tons)	374,330 lb. (167 tons)	336,290 lb. (150 tons)	6.04	7031	868,500	868,500	868,500

APPENDIX I

III

Similar properties of certain octagonal piles are given in the next three tables. In calculating the strengths of the piles the following data were used.

Width of pile across the flats (in.)	12	15	18
Area (sq. in.)	119	186	268
Longitudinal steel,	3 per cent. of the total area		
say	Eight $\frac{3}{4}$ -in. bars	Eight $\frac{5}{8}$ -in. bars	Eight $\frac{1}{2}$ -in. bars
Transverse steel,	1 per cent. of the total volume		
say, $\frac{3}{8}$ -in. spiral at pitch of	2 $\frac{3}{4}$ in.	2 $\frac{1}{2}$ in.	2 in.
Core diameter (in.)	9 $\frac{1}{4}$	12 $\frac{1}{4}$	15 $\frac{1}{4}$

12-IN. OCTAGONAL PILES.

(Cross-sectional area = 119 sq. in. Weight per foot run = 119 lb.)

Reinforce- ment	Short-column load			Depth of neutral axis (in.)	Moment of inertia (in. ⁴)	Safe moment of resistance (in.-lb.)		
	1:1:2	1:1 $\frac{1}{2}$:3	1:2:4			1:1:2	1:1 $\frac{1}{2}$:3	1:2:4
Eight $\frac{3}{4}$ -in. bars	163,470 lb. (73 tons)	149,580 lb. (67 tons)	135,690 lb. (61 tons)	4.52	814	180,460	180,460	171,200

15-IN. OCTAGONAL PILES.

(Cross-sectional area = 186 sq. in. Weight per foot run = 186 lb.)

Reinforce- ment	Short-column load			Depth of neutral axis (in.)	Moment of inertia (in. ⁴)	Safe moment of resistance (in.-lb.)		
	1:1:2	1:1 $\frac{1}{2}$:3	1:2:4			1:1:2	1:1 $\frac{1}{2}$:3	1:2:4
Eight $\frac{1}{2}$ -in. bars	255,430 lb. (114 tons)	233,720 lb. (105 tons)	212,020 lb. (95 tons)	5.65	2140	362,820	362,820	362,820

18-IN. OCTAGONAL PILES.

(Cross-sectional area = 268 sq. in. Weight per foot run = 268 lb.)

Reinforce- ment	Short-column load			Depth of neutral axis (in.)	Moment of inertia (in. ⁴)	Safe moment of resistance (in.-lb.)		
	1:1:2	1:1 $\frac{1}{2}$:3	1:2:4			1:1:2	1:1 $\frac{1}{2}$:3	1:2:4
Eight $\frac{1}{2}$ -in. bars	367,810 lb. (164 tons)	336,560 lb. (150 tons)	313,300 lb. (140 tons)	6.78	4658	639,260	639,260	639,260

Note.—The cross-sectional area of an octagon is $0.8284d^2$, where d is the width of the octagon measured over the flats.

APPENDIX II

Safe Lengths for Lifting certain Reinforced Concrete Piles

THE Table gives the maximum lengths of some of the square and octagonal piles listed in Appendix I which can be supported at their extreme ends without exceeding the safe moment of resistance given there.

SQUARE PILES MADE WITH HIGH-GRADE CONCRETE 1 : 1½ : 3.

									Maximum length.
12-in. pile	1-in. bars .	.	.	.	.	.	.	.	33 ft. 2 in.
" "	1¼-in. " .	.	.	.	.	.	.	.	40 ft. 9 in.
" "	1½-in. " .	.	.	.	.	.	.	.	46 ft. 9 in.
14-in. pile	1-in. bars .	.	.	.	.	.	.	.	31 ft. 9 in.
" "	1¼-in. " .	.	.	.	.	.	.	.	40 ft. 2 in.
" "	1½-in. " .	.	.	.	.	.	.	.	46 ft. 0 in.
15-in. pile	1-in. bars .	.	.	.	.	.	.	.	30 ft. 10 in.
" "	1¼-in. " .	.	.	.	.	.	.	.	37 ft. 10 in.
" "	1½-in. " .	.	.	.	.	.	.	.	45 ft. 0 in.
16-in. pile	1-in. bars .	.	.	.	.	.	.	.	30 ft. 0 in.
" "	1¼-in. " .	.	.	.	.	.	.	.	37 ft. 2 in.
" "	1½-in. " .	.	.	.	.	.	.	.	43 ft. 11 in.
18-in. pile	1-in. bars .	.	.	.	.	.	.	.	28 ft. 10 in.
" "	1¼-in. " .	.	.	.	.	.	.	.	35 ft. 8 in.
" "	1½-in. " .	.	.	.	.	.	.	.	42 ft. 3 in.

OCTAGONAL PILES MADE WITH HIGH-GRADE CONCRETE 1 : 1½ : 3.

12-in. pile .	.	.	.	.	.	.	.	.	31 ft. 9 in.
15-in. pile .	.	.	.	.	.	.	.	.	36 ft. 0 in.
18-in. pile .	.	.	.	.	.	.	.	.	39 ft. 10 in.

APPENDIX III

Forces and Bending Moment induced in certain Reinforced Concrete Retaining Walls

THIS table gives the results of calculations of the forces induced in retaining walls of various heights and of the type shown in *Fig. 22*, page 36.

In each case it is assumed that the earth behind and beneath the wall is a sand ($w = 0.056$ tons per cubic foot ; $\phi = 30$ deg.).

1	2	3	4	5	6	7	8
Height from lower ground to upper ground (ft.)	Depth of tie below upper ground (ft.)	Lateral pressure per foot run of wall <i>P</i> (tons)	Lateral resistance per foot run of wall <i>R</i> (tons)	Depth of pile below lower ground level <i>D</i> (ft.)	Pull on anchor ties per foot run of wall <i>T</i> (tons)	Bending moment per foot run of wall (in.-lb.)	Negative bending moment at tie (in.-lb.)
10	nil	1.77	1.30	4.01	0.47	59,400	0
	2½	1.70	1.13	3.74	0.57	42,600	1,260
	5	1.57	0.84	3.22	0.73	20,000	10,080
20	nil	7.06	5.20	8.02	1.86	478,000	0
	2½	6.95	4.91	7.78	2.04	414,000	1,260
	5	6.80	4.53	7.48	2.27	341,000	10,080
	7½	6.59	4.04	7.06	2.55	256,000	34,020
	10	6.29	3.36	6.44	2.93	161,000	80,640
30	nil	15.90	11.72	12.03	4.18	1,608,000	0
	5	15.52	10.77	11.53	4.75	1,317,000	10,080
	10	15.00	9.50	10.83	5.50	959,000	80,640
	15	14.15	7.55	9.65	6.60	542,000	272,160

Effect of variations of nature and consistency of backing.

In this table the earth pressure and resistance have been calculated from Rankine's formula assuming that the upper ground surface is level, and that the earth weighs 125 lb. per cubic foot and has an angle of repose of 30 deg. Although Rankine's formula is no doubt incorrect the results obtained are probably not far amiss for an ordinary dirty moist gravel or sand such as commonly occurs in nature. The lateral pressure thus calculated is equivalent to that of a fluid weighing 41 lb. per cubic foot. Of course, with different backings the pressure may be very much less or very much greater than this. For instance a very clean broken stone might exercise a fluid pressure less than half this amount, and on

the other hand a very soft wet clay might exercise a pressure of more than double it. The research work which is being carried out in this and other countries will probably change our methods of estimating lateral pressure and resistance, but meanwhile it may be mentioned that a summary of common British practice is contained in the Report on Gravity Retaining Walls issued by the Institution of Structural Engineers.

In cases where the backing to a retaining wall has to be imported, it is obviously worth while, even at some additional expense, to select a material such as ashes, broken stone, clean gravel, or clean coarse sand, which has a high coefficient of friction undiminished by the percolation of water. Ashes are probably best of all, as they are low in weight and high in friction.

APPENDIX IV

Stresses during Driving

THE "Notes" referred to in Chapter II are as follows :

(1) THE NATURE OF THE STRESSES.—The wave-theory of the propagation of stress may be applied. The compression due to the blow travels from the head at a velocity of about 12,200 ft. per second and is reflected from the foot as a compression or a tension according to whether driving is hard or easy. The stress at any point is the sum of the stresses due to the down- and up-travelling waves. Under conditions of hard driving compressive stresses may exceed 3000 lb. per square inch.

(2) HEAD-CONDITIONS.—The cushion at the head of the pile, that is, the dolly and the packing in the helmet, plays an important part in determining the stresses ; the softer the cushion the lower the maximum stress. For a cushion with a linear stress-strain relation the stiffness-constant (k/A) is the stress on the pile-head to produce unit compression. Cushions usually have a non-linear stress-strain relation, and therefore k/A must be defined as "at . . . lb. per square inch." At 3000 lb. per square inch values of k/A range, in practice, from 10,000 to 40,000 lb. per square inch, and at 2000 lb. per square inch from 6670 to 26,700 lb. per square inch, k/A being approximately proportional to stress. Most forms of packing harden during driving. With piles of length greater than 30 ft., the maximum stress at the head is generally independent of the conditions at the foot of the pile and may be estimated from considerations of hammer weight and drop, cushion constants, and pile design and dimensions only.

(3) FOOT-CONDITIONS.—For very easy driving conditions, that is, with very large sets, the compressive stresses at the toe will be very low and the stress-wave will be reflected as a tension, which when combined with the down-coming compression-wave produces tensions which increase from zero at the toe to a maximum towards the middle of the pile. No failures due to these tensile stresses have been observed. As resistance at the toe increases the compressive stress increases, and may theoretically reach twice the value of the maximum head-stress. Values 50 per cent. greater have been recorded.

The foot-stresses depend on the total movement of the toe, that is, the set as ordinarily measured and the earth movement at the toe. For the purpose of stress estimation the ordinary or permanent set has been termed the "plastic" set and the earth movement the "elastic" set. When combined, as follows, they have been called the "equivalent elastic set."

Equivalent elastic set = twice plastic set (or permanent set as ordinarily measured) + elastic set (or earth-movement).

The worst conditions are obtained where the whole of the resistance to penetration is concentrated at the toe, and the theory and charts given are based on this assumption. Friction at the sides of the pile will have only a small effect on head-stresses, but may have an important influence in reducing the stresses below ground.

(4) MEASUREMENT OF EQUIVALENT ELASTIC SET.—The set-recorder is shown

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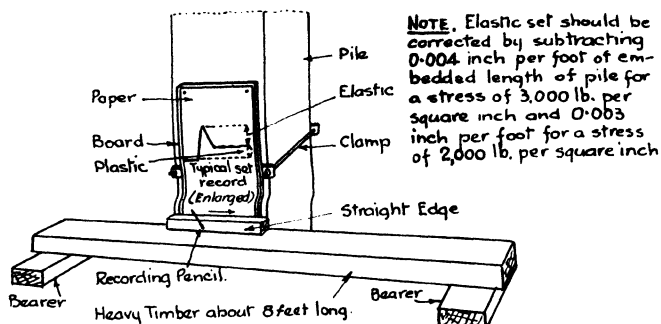


Fig. 80.—Set Recorder.

in Fig. 80 together with a typical record. A correction to the elastic set is necessary to allow for the elastic compression in the pile. This is 0.004 in. per foot of pile embedded where the maximum head-stress is 3000 lb. per square inch, and 0.003 where the stress is 2000 lb. per square inch. Further investigation of the order of the elastic and plastic sets occurring in practice is required.

(5) ESTIMATING STRESSES.—A series of charts is given with this Note, enabling any particular piling conditions to be examined to ascertain whether maximum stresses of 3000 lb. per square inch or 2000 lb. per square inch are likely to be exceeded during driving. Three conditions of head cushion have been included, namely, soft,

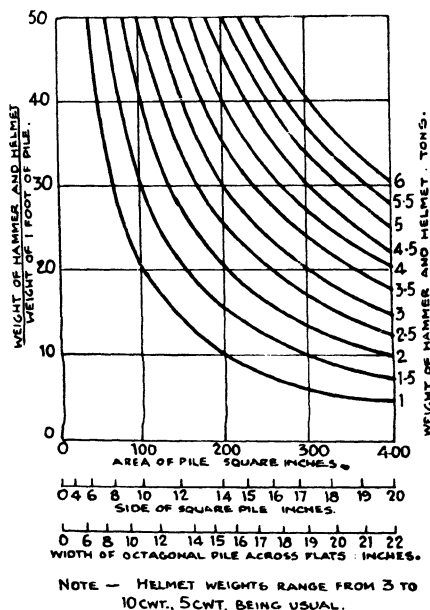
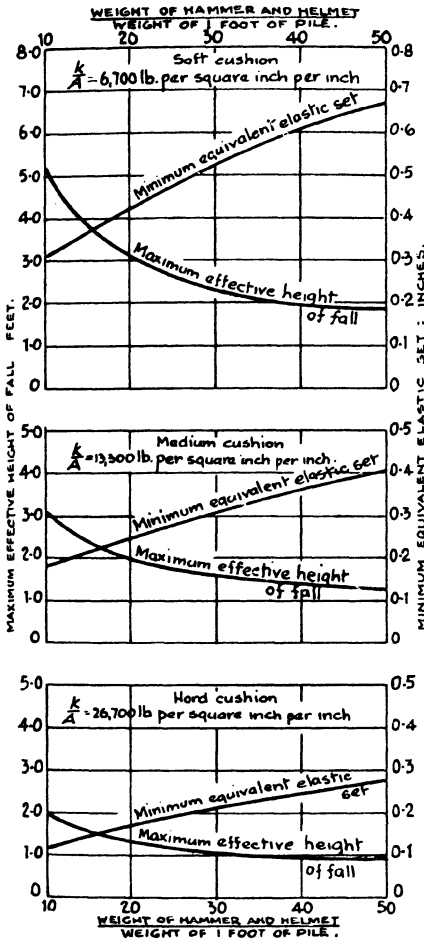


Fig. 81.—Diagram giving the Ratio $\frac{\text{Weight of Hammer and Helmet}}{\text{Weight of 1 foot of Pile}}$ for a Range of Pile-Sizes and Hammer-Weights.
(Weight of reinforced concrete taken as 160 lb. per cubic foot.)

(a) FOR MAXIMUM STRESS OF 2,000 LB. PER SQUARE INCH



(b) FOR MAXIMUM STRESS OF 3,000 LB. PER SQUARE INCH.

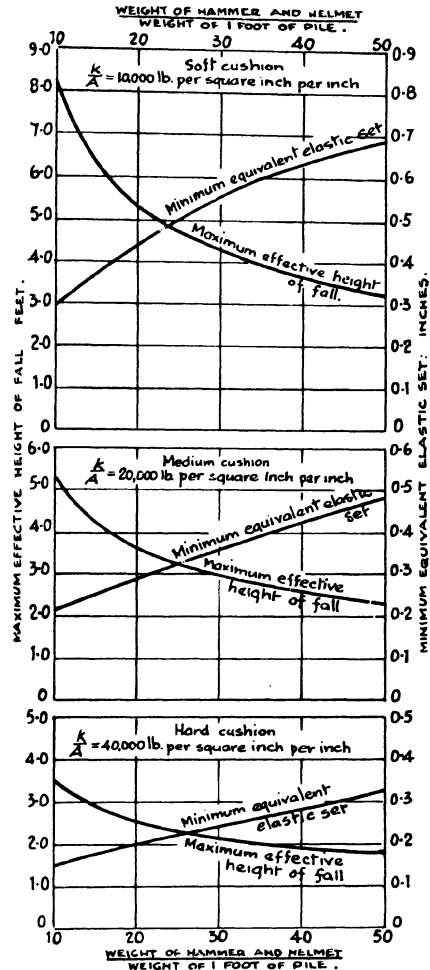


Fig. 82.—Relation between the Ratio $\frac{\text{Weight of Hammer and Helmet}}{\text{Weight of 1 foot of Pile}}$ and the Effective Height of Fall and Minimum Equivalent Set for a given Maximum Stress in the Pile.

($2\frac{1}{2}$ per cent. longitudinal reinforcement in pile. Young's Modulus for concrete 4.5×10^6 lb. per square inch.)

medium, and hard. For all types of packing tested the hard condition has been found to apply after about 1000 blows.

From Fig. 81 the ratio $\frac{\text{weight of hammer and helmet}}{\text{weight of 1 foot of pile}}$ is first obtained.

From Fig. 82 (a) or 82 (b), depending on whether 2000 lb. or 3000 lb. per square inch is selected as a maximum for working conditions, the effective height of fall

is then obtained for the particular conditions of head cushion required. This effective height of fall is then converted to the height of free fall by means of *Fig. 83*. Any height of fall greater than this will produce a head stress greater than that selected.

Figs. 82 (a) and *82 (b)* also enable the equivalent elastic set which produces a similar stress at the toe, that is, either 2000 lb. or 3000 lb. per square inch, to be obtained. Equivalent elastic sets lower in value and falling below the curve produce higher stresses.

(6) HAMMER-WEIGHT IN RELATION TO PILE-SIZE, ETC.—The best conditions of driving are obtained by using the heaviest possible hammer together with the softest head cushion (lowest stiffness k/A), the height of drop being adjusted to give

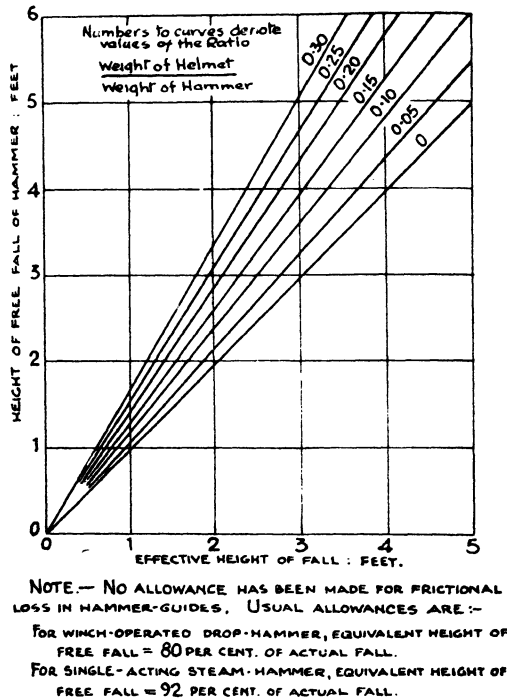


Fig. 83.—Conversion of Effective Height of Fall to Height of Free Fall of Hammer for Various Ratios $\frac{\text{Weight of Helmet}}{\text{Weight of Hammer}}$.

a safe stress. It is suggested that a reasonable minimum value of the ratio $\frac{\text{weight of hammer}}{\text{weight of 1 foot of pile}}$ would be 30. This gives for 12-in., 14-in., 16-in., and 18-in. square piles, hammers of $2\frac{1}{2}$, 3, $3\frac{3}{4}$, and $4\frac{3}{4}$ tons respectively (see *Fig. 81*).

In nearly all cases the equivalent elastic set increases practically in proportion to the hammer weight, and experimental evidence shows that the plastic set (set as ordinarily measured) increases at a greater rate.

(7) PEAK-STRESS INDICATOR.—The head stress may be determined from the peak-stress indicator * which is attached to the hammer and measures its deceleration. Alternatively, the instrument may be used to indicate when any predetermined

* See Journal of Inst. C.E., December 1935, p. 158.

value is exceeded. Measurement of the elastic and plastic sets enables the stress values thus determined to be used to obtain foot stresses. The charts already given may be used for this purpose where the indicator is adjusted to 2000 lb. or 3000 lb. per square inch. Further experience of the working of the indicator in practice is necessary.

(8) **IMPACT STRENGTH OF CONCRETE AND REINFORCED CONCRETE.**—The impact strength of concrete may be as low as 50 per cent. of the crushing strength. For a working stress of 3000 lb. per square inch a concrete of crushing strength not less than 6000 lb. per square inch is therefore necessary, and for 2000 lb. per square inch not less than 4000 lb. per square inch.

To obtain strengths greater than 6000 lb. per square inch proportions not leaner than 1 : 1½ : 3 (nominal), that is, 1 cwt. of cement to 1½ cb. ft. of sand and 3½ cb. ft. of coarse aggregate, should be used, and the greatest care exercised in the selection of aggregates, the control of water content, and curing. (It is of interest to note that a crushing strength of only 3300 lb. per square inch is required for 1 : 1½ : 3 High-grade concrete under the Recommendations for a Code of Practice.) For easier driving conditions, where the lower crushing strength is adequate, that strength might be obtained by careful control with a 1 : 2 : 4 mix.

Curing conditions have a very marked effect on impact strength, and piles should be cured under damp conditions as long as practicable. Unless conditions of driving are easy it is recommended that this period should be not less than 14 days. Further information is required on impact strength and on the factors influencing it.

Longitudinal reinforcement does not affect the impact strength greatly. The amount of lateral reinforcement, on the other hand, profoundly affects the impact resistance of a pile, particularly at the head and toe. It is recommended that for a length from the extremities of 2½ to 3 times the external diameter of the pile the volume of lateral reinforcement should not be less than 1 per cent. of that of the gross volume of the corresponding length of pile. The diameter of the ties should conform with the usual practice for reinforced concrete, and should be not less than $\frac{3}{16}$ in. or one-fourth of the diameter of the main bars, whichever is the greater. The minimum spacing of the ties at head and foot should be such as to provide ample facility for placing the concrete. It was observed on an outside contract that the performance of piles reinforced with heavy spirals (2½ per cent.) was definitely good, and although patches of surface spalling occurred they did not materially affect the resistance of the pile to further driving.

External head bands placed in the mould before casting the concrete considerably strengthen the pile head.

(9) **IMPROVEMENTS IN HEAD-CUSHION.**—To put pile driving on a proper scientific basis an improved form of head-cushion is required possessing the qualities of permanence and of low and constant stiffness. No entirely satisfactory helmet-packing has yet been discovered, and it is possible that a mechanical device to take the place of the dolly, helmet, and packing will afford the most satisfactory solution.

(10) **NECESSITY FOR CARE IN DETAILS.**—The margin of safety in driving reinforced concrete piles is frequently so low that slight carelessness in the manufacture and driving of the pile may be sufficient to cause failure. Care should be taken to restrict the water content of the mix so far as is compatible with the production of a thoroughly dense and well compacted concrete. The head of the pile should be carefully formed, and all surfaces in the helmet should be truly plane and at right angles to the axis of the pile. It is most important that the helmet packing should be placed evenly on the pile head, and that the layer immediately in contact with the head should be of soft material covering the whole surface. The fall of the hammer should be parallel with the long axis of the pile, and the blow should be delivered as nearly concentrically as possible.

(II) BEARING CAPACITY.—The dependence of the set produced on the packing conditions indicates the importance of specifying the condition and nature of the packing to be used in determining the sets on which bearing capacity is estimated. Failing a standard packing it should at least be specified that the packing shall be well compacted, thus ensuring the maximum set per blow. Up to the present the research has not been concerned with the bearing capacity of piles as such.

Method of Using Diagrams.

Given the dimensions of the pile and the weight of hammer and helmet, to determine the maximum actual height of drop of hammer to produce either 2000 lb. or 3000 lb. per square inch maximum stress in the concrete :

(a) From *Fig. 81* determine the ratio

$$\frac{\text{weight of hammer and helmet}}{\text{weight of 1 ft. of pile}}$$

(b) From *Fig. 82 (a)* for 2000 lb. per square inch maximum stress, or *Fig. 82 (b)* for 3000 lb. per square inch, determine the maximum effective height of fall for the appropriate cushion (the hard cushion and therefore the highest value of k/A applies after about 1000 blows).

(c) From *Fig. 83* determine the height of free fall for the appropriate ratio

$$\frac{\text{weight of helmet}}{\text{weight of hammer}}$$

(d) To obtain the actual height of drop, divide this value by 0.8 for a winch-operated drop-hammer, or 0.92 for a single-acting steam hammer.

(e) From *Fig. 82 (a)* or *82 (b)* determine the minimum equivalent elastic set (see *Fig. 80*) to produce a foot-stress of 2000 lb. or 3000 lb. per square inch respectively.

The set recorder referred to in paragraph (4) and shown in *Fig. 80* consists of a board carrying a sheet of paper and which is firmly fixed to the front face of the pile by clamps. A straightedge resting against the board is nailed to a heavy baulk of timber which is carried by two bearers one on each side of the pile. A pencil drawn by hand along the straightedge traces the record from which the amounts of both the elastic (temporary) and plastic (permanent) sets can be measured, and hence the "equivalent-elastic set" referred to in paragraph (3).

APPENDIX V

Standard Method of Making Works Cube Tests of Concrete

THE method described in the Recommendations for a Code of Practice for Reinforced Concrete in Building applies to compression tests of concrete sampled during the progress of the work.

SIZE OF TEST CUBES AND MOULDS.—The test specimens shall be 6-in. cubes. The moulds shall be of steel or cast iron, with inner faces accurately machined in order that opposite sides of the specimen shall be plane and parallel. Each mould shall be provided with a base plate having a plane surface and of such dimensions as to support the mould during filling without leakage and preferably attached by springs or screws to the mould. Before placing the concrete in the mould both the base plate and the mould shall be oiled to prevent sticking of the concrete.

SAMPLING OF CONCRETE.—Wherever practicable concrete for the test cubes shall be taken immediately after it has been deposited in the work. Where this is impracticable samples shall be taken as the concrete is being delivered at the point of deposit, care being taken to obtain a representative sample. All the concrete for each sample shall be taken from one place. A sufficient number of samples, each large enough to make one test cube, shall be taken at different points so that the test cubes made from them will be representative of the concrete placed in that portion of the structure selected for tests. The location from which each sample is taken shall be noted clearly for future reference.

In securing samples the concrete shall be taken from the mass by a shovel or similar implement and placed in a large pail or other receptacle for transporting to the place of moulding. Care shall be taken to see that each test cube represents the total mixture of concrete from a given place. Different samples shall not be mixed together but each sample shall make one cube. The receptacle containing the concrete shall be taken to the place where the cube is to be moulded as quickly as possible and the concrete shall be slightly re-mixed before placing in the mould.

CONSISTENCY.—The consistency of each sample of concrete shall be measured, immediately after re-mixing, by the slump test made in accordance with the Method of Test for Consistency of Concrete recommended by the Building Research Station. Providing that care is taken to ensure that no water is lost, the material used for the slump tests may be re-mixed with the remainder of the mix before making the test cube.

COMPACTING.—Concrete test cubes shall be moulded by placing the fresh concrete in the mould in three layers, each layer being rammed with a steel bar 15 in. long and having a ramming face of 1 in. square and a weight of 4 lb. For mixes of 1½ in. slump or less, 35 strokes of the bar shall be given for each layer; for mixes of wetter consistency the number may be reduced to 25 strokes per layer.

CURING.—The test cubes shall be stored at the site of construction, at a place free from vibration, under damp sacks for 24 hours ($\pm \frac{1}{2}$ hour), after which time they shall be removed from their moulds, marked, and buried in damp sand until the

REINFORCED CONCRETE PILING

time for sending to the testing laboratory. They shall then be well packed in damp sand or other suitable damp material and sent to the testing laboratory, where they shall be similarly stored until the date of test. Test cubes shall be kept on the site for as long as practicable and for at least three-fourths of the period before test except for tests at ages less than seven days.

The temperature of the place of storage on the site shall not be allowed to fall below 40 deg. F. nor shall the cubes themselves be artificially heated.

RECORD OF TEMPERATURES.—A record of the maximum and minimum day and night temperatures at the place of storage of the cubes shall be kept during the period the cubes remain on the site.

METHOD OF TESTING.—All compression tests on concrete cubes shall be made between smooth plane steel plates without end packing, the rate of loading being kept approximately at 2000 lb. per square inch per minute. One compression plate of the machine shall be provided with a ball seating in the form of a portion of a sphere, the centre of which falls at the central point of the face of the plate. All cubes shall be placed in the machine in such a manner that the load shall be applied to the sides of the cubes as cast.

APPENDIX VI

Slump Test.

THE test is to be used both in the laboratory and during the progress of the work for determining the consistency of concrete.

The test specimen shall be formed in a mould in the form of the frustrum of a cone with internal dimensions as follows: Bottom diameter 8 in., top diameter 4 in., height 12 in. The bottom and the top shall be open, parallel to each other, and at right angles to the axis of the cone. The mould shall be provided with suitable foot pieces and handles. The internal surface should be smooth.

Care shall be taken to ensure that a representative sample is taken.

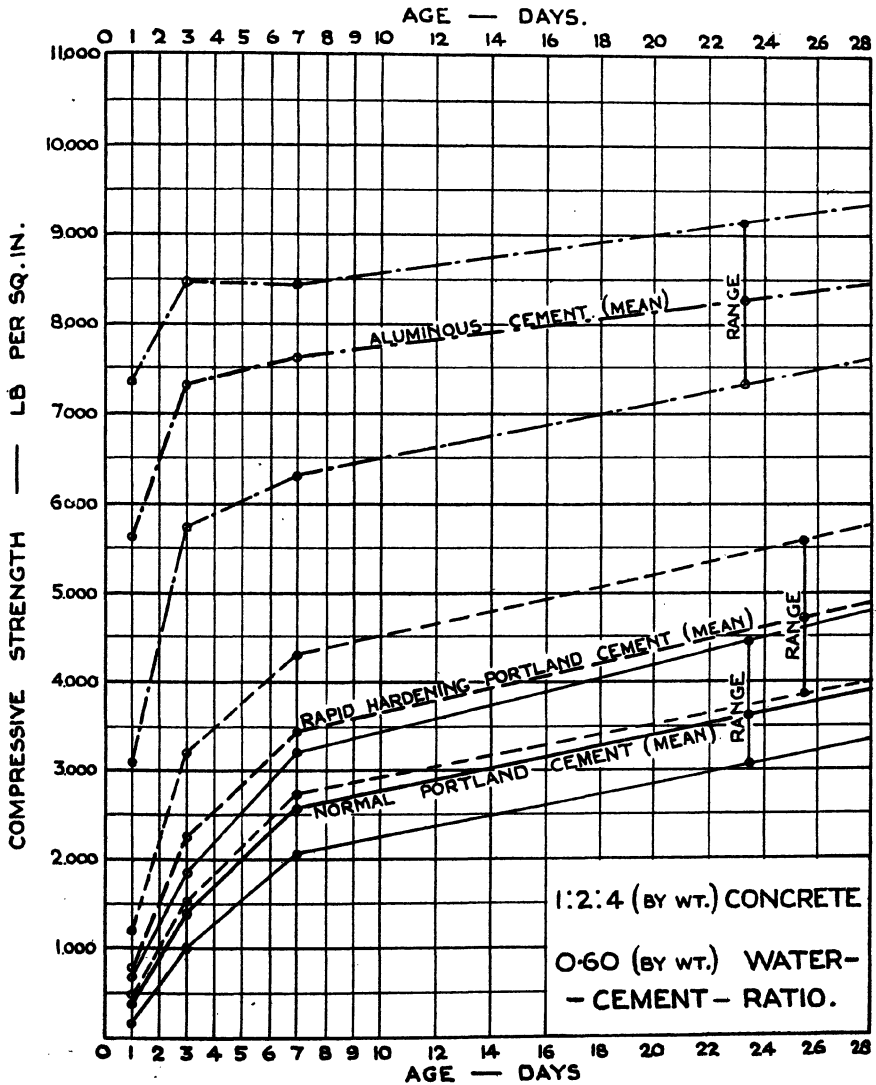
The internal surface of the mould shall be thoroughly clean, dry, and free from set cement before commencing the test.

The mould shall be placed on a smooth, flat, non-absorbent surface, and the operator shall hold the mould firmly in place, while it is being filled, by standing on the foot pieces. The mould shall be filled to about one-fourth of its height with the concrete which shall then be puddled, using 25 strokes of a $\frac{3}{8}$ -in. rod, 2 ft. long, bullet pointed at the lower end. The filling shall be completed in successive layers similar to the first and the top struck off so that the mould is exactly filled. The mould shall then be removed by raising vertically, immediately after filling. The moulded concrete shall then be allowed to subside and the height of the specimen measured after coming to rest.

The consistency shall be recorded in terms of inches of subsidence of the specimen during the test, which shall be known as the slump.

APPENDIX VII

Age-Strength Curves for Concrete



ACKNOWLEDGMENTS.

WE are indebted to the following for the illustrations mentioned.

Fig. 31.—Mr. G. Ellson, O.B.E., Chief Engineer, Southern Railway.

Fig. 32.—Messrs. Mott, Hay & Anderson.

Figs. 33, 34, 38, 39, 73, 74, 75.—Mr. E. H. Buckley, Southampton.

Fig. 35.—The British Reinforced Concrete Engineering Co., Ltd.

Fig. 36.—Mr. T. Hardie, Chief Engineer of The Gas Light & Coke Co., and Concrete Piling, Ltd.

Fig. 37.—Messrs. Samuel Williams & Co., Ltd.

Fig. 40.—Messrs. R. Robinson & Co. (Contractors), Ltd.

Fig. 41.—Messrs. John Mowlem & Co., Ltd.

Fig. 43.—Messrs. J. Shelbourne & Co., Ltd.

Figs. 42, 44, 46, 47, 52, 79.—The British Steel Piling Co., Ltd. (*Fig. 79 is reproduced, by permission, from a drawing published in "Engineering."*)

Fig. 51.—West's Rotinoff Piling & Construction Co., Ltd.

Fig. 53.—Franki Compressed Pile Co., Ltd.

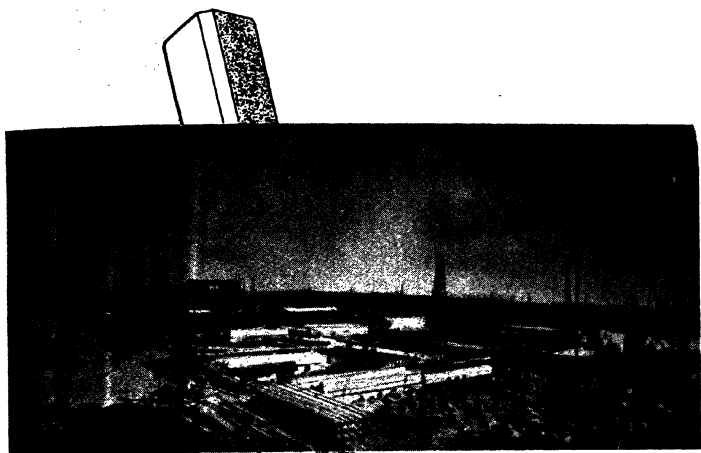
Figs. 54–59.—The Pressure Piling Co., Ltd.

Appendix VII.—“Explanatory Handbook on the Code of Practice for Reinforced Concrete,” by W. L. Scott and W. H. Glanville (Concrete Publications, Limited. Price 8s.).

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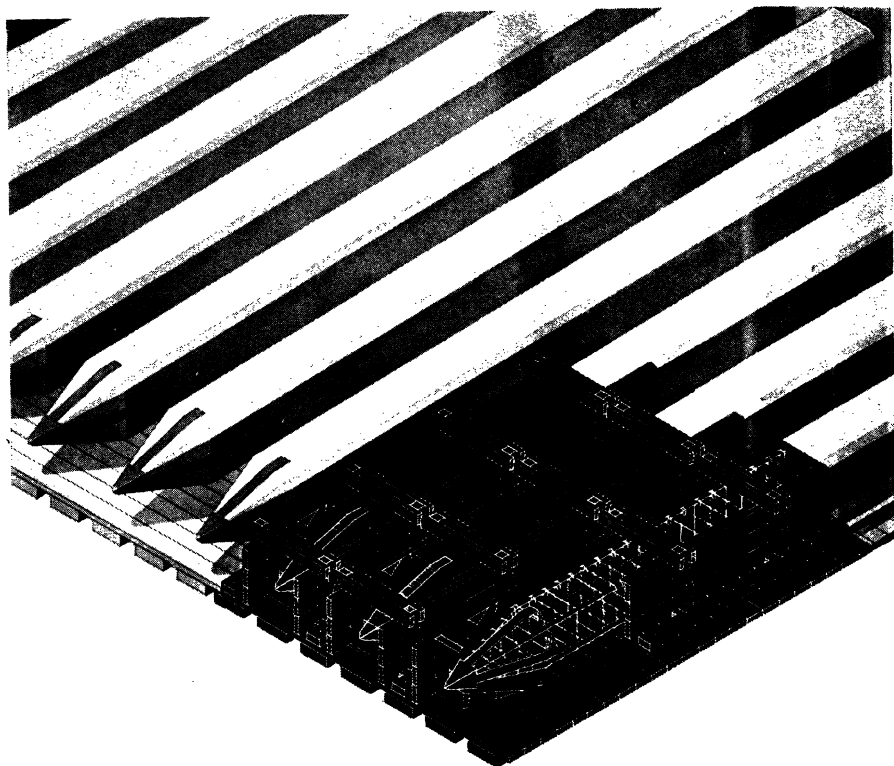
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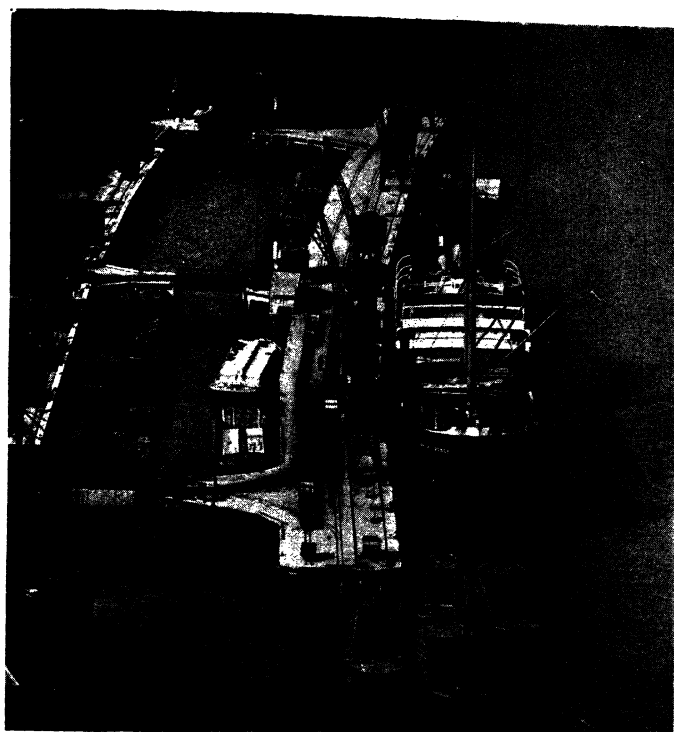
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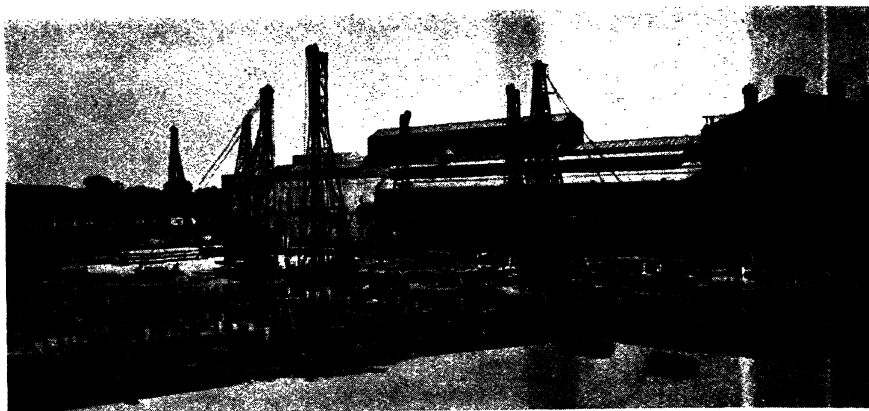
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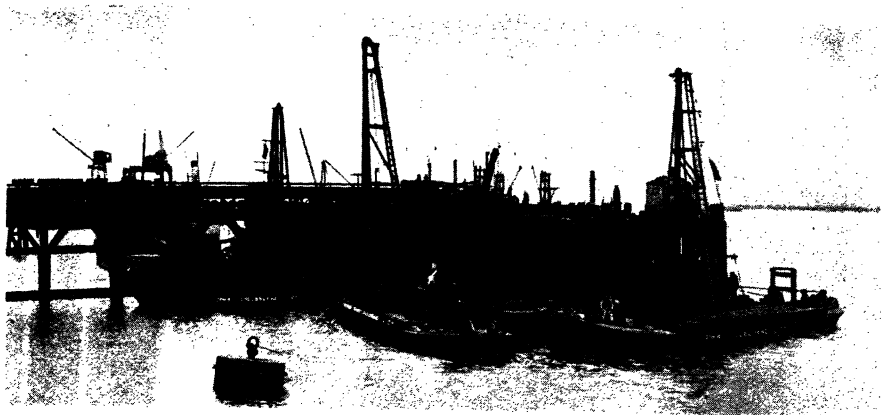
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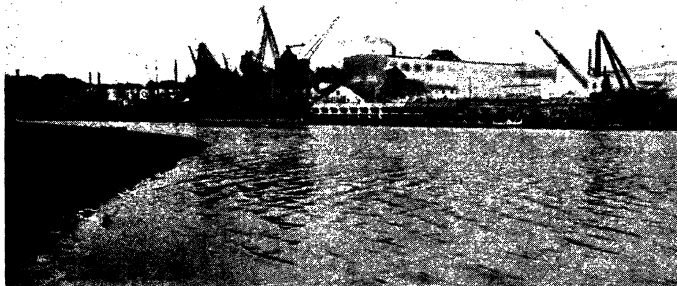
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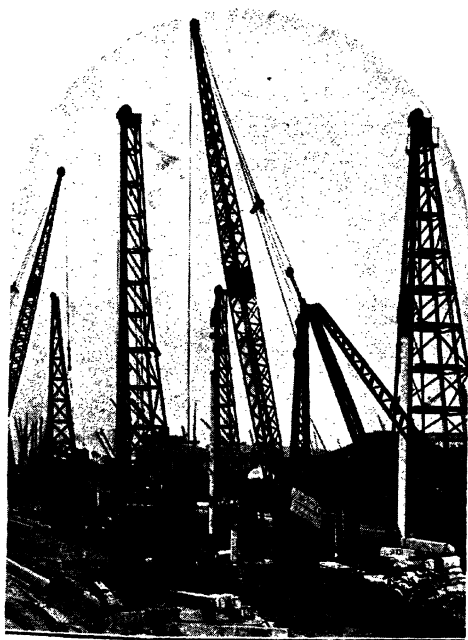
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